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ELEVATED REINFORCED-CONCRETE WATER TANKS IN CUBA.

In the accompanying half-tones and drawings there are shown a pair of reinforced-concrete water tanks recently built near Havana, Cuba, which, though possessing nothing very extraordinary either in design or size, have some unusual features that make them worthy of description. The tanks, shown completed in Fig. 1, are entirely separate except for a walk which joins the two structures at the bottom of the tanks proper. Each portion consists of a circular reinforced-concrete tank, holding 129,000 gals., and 24 ft. 2 ins. high and 33 ft. 8 ins. in outside diameter, carried on a double row of arch-connected columns, so that the tops of the tanks are 62 ft. 8 ins. from the ground. In each tank the walls are 12 ins. thick, plumb on both sides and reinforced in both directions by ½-in. rods spaced 6 ins. c. to c. The top of the tank is a flat 3-in. slab reinforced with wire mesh and resting on crossed beams and girders, but the bottom is of an entirely different and novel construction. It is a spherical dome with a base of 30 ft. in diameter and a rise of 5 ft., reinforced radially by ⅝-in. rods both at extrados and intrados and circumferentially by 1½-in. rods spaced about 2 ft. on centers. The thrust at the circumference is cared for by a circular beam heavily reinforced as shown in Fig. 2. Fig. 4 will serve to illustrate the method of placing the reinforcement in the tank bottom.

The remaining details of construction are standard and can be readily understood from the drawings in Figs. 2 and 3. No waterproofing precautions were taken other than to lay a thoroughly mixed wet concrete, and the tanks are now satisfactorily holding the required supply of water.

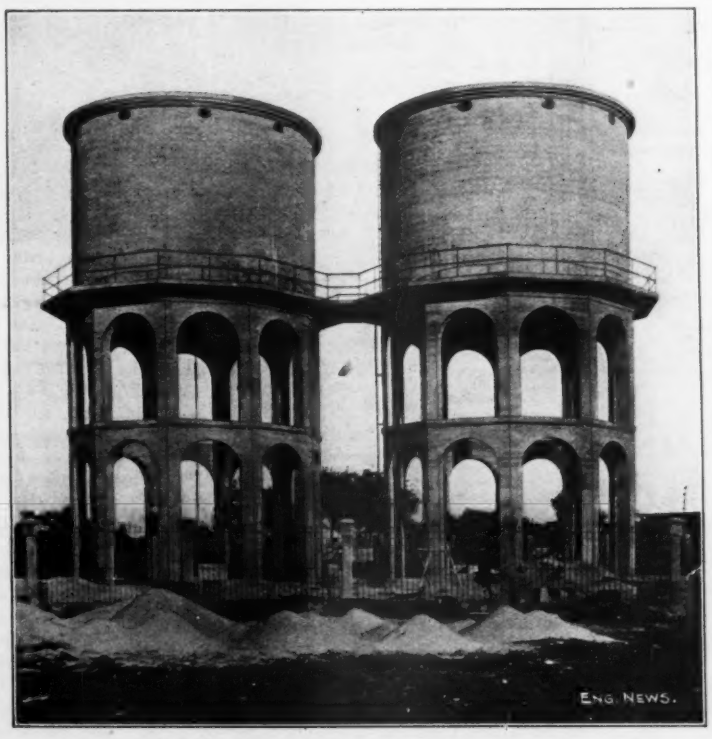

FIG. 1. REINFORCED-CONCRETE WATER TANKS JUST OUTSIDE OF HAVANA, CUBA.

The tanks are part of the supplementary water system of the city of Havana, and are located at Marianao. They were built by Senor Esteban Duque Estrada, City Engineer of Havana, under the supervisory counsel of Col. William M. Black, Corps of Engineers, U. S. A., who, since the so-called "revolution" of 1906 and the subsequent Provisional Government, has been "Advisor" to the Board of Public Works. The Snare & Triest Co., of New York City, were the contractors. Mr. Henry A. Young, Assoc. M. Am. Soc. C. E., now Engineer of the Camagüey, Cuba, waterworks, was immediately in charge of the design, and we are indebted to him for the information and illustrations herewith presented.

THE USE AND ABUSE OF COST DATA.

By W. W. PATCH,* M. Am. Soc. C. E.

The subject of cost data has been of somewhat more than ordinary interest during the past few months, owing largely to the efforts which have been made by a portion of the press and technical journals to prove that the accepted bid for building a series of large earthen dikes is much greater than the probable cost of actual construction. Some writers even have ventured to assert that in estimating the probable cost of work, with a view to bidding, exact results can be obtained by building up synthetically from observed detailed costs of similar work collected from various and scattered sources. Published cost data are no doubt worthy of considerable attention, but their value has been much exaggerated, and the conservative engineer will not rely very strongly upon them unless he has first-hand information upon many points not usually covered by such reports.

In the compilation of cost data there are generally many omissions. Some of the most flagrant of which may be noted below:

(a) Lack of uniformity in arrangement, or entire failure to make any logical arrangement of the several minor operations necessary in constructing a given piece of work; as for example, in rolled embankment, the work might be separated into excavating, hauling, spreading, watering and rolling; or in making concrete the cost could be subdivided into that of materials at mixer, mixing, transporting concrete, placing forms, miscellaneous.

(b) Failure to note rates of wages, length of working day, and cost of supplies (fuel, oil, etc.).

(c) Failure to note efficiency of labor, and condition of the labor market as affecting the supply of laborers.

(d) Omission of statements regarding climatic or topographic conditions as agencies hindering or expediting progress.

(e) The very general disregard of charges for depreciation and repairs to plant, interest on investment, and general expense.

(f) Ignoring the effect which the time limit specified has upon amount of equipment required and depreciation charges for same; and disregarding special requirements of the specifications as they increase or diminish the cost of work.

(g) The failure to separate the costs of an item into allowances for labor, supplies, plant and repair charges, etc.

(h) The failure to describe the different materials encountered, and the rate of progress possible in each.

The foregoing list might be continued almost indefinitely, but perhaps enough items have been listed to show the danger confronting the inexperienced engineer, who hopes to make a close estimate on prospective work by relying solely upon published cost data.

As an example of the practical utility of the average cost data let us consider some recently published detailed costs on concrete construction. In this particular case about 72,000 cu. yds. of heavy concrete paving and cut-off walls were to be built in two years in a dam across a large river, subject to great and long continued floods which put a stop to work during three months each year. While the article under discussion is not entirely clear on this point, yet it is to be inferred that all sand and gravel for concrete would be obtained by crushing rock, under which conditions a contractor would have to figure on supplying a crushing, screening, and concrete mixing plant having a capacity of from 5,000 to 6,000 cu. yds. per month, and as the work was located in a desert, 12 miles from a railroad, it

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is to be assumed that the cost of transporting and installing this plant would be considerable, and that the depreciation charge would be correspondingly large.

The table of cost data for this concrete contains 28 items, showing a total cost of $6.38 per cu. yd., without cement, for one month's work; the concrete being machine-mixed and hauled 1,100 ft. The total charge for equipment, depreciation and repairs is 2.8%, or 18 cts. per cu. yd., which on the total of 72,000 cu. yds. would represent but $13,000 for entire plant depreciation and repairs for the two years' job. As the plant included derricks, hoisting engines, track and cars, used in placing the concrete, in addition to the necessary crushing, screening, and mixing machinery, the depreciation charges certainly look too small.

By rearranging and grouping the various items in the tabulation, the itemized costs per cu. yd. given in the article read as follows:

$0.56—Labor (11 items).
0.18—Depreciation and repairs.
0.08—Fuel, oil, light and water.
0.91—"Hauling concrete and rock" (1 item).
4.65—"Concrete" (1 item).

$6.38—Total.

Suppose that a bid were being prepared upon other work where many of the conditions were similar. Among other things, the contractor would wish to know the rates of wages, and the cost of the following items: Sand and stone at screening plant, hauling sand and stone to mixer, mixing concrete, transporting concrete, forms, sprinkling concrete, and hauling and caring for cement. The tabulation discussed does not furnish data in the proper shape to determine any one of these items, and therefore fails very largely in its usefulness.

To illustrate how one may be misled in using cost data without a full knowledge of the conditions to which they apply, a specific instance which has come under the writer's observation may be mentioned. In some recent work two methods were in use in making a rolled earthen embankment, in one of which the earth is hauled to the embankment in wagons, and in the other by cars. The spreading of the earth in the latter case has cost seven times what it did in the former. Yet in a recently published estimate of probable cost of making an embankment which was admittedly to be done by car work, an allowance for spreading was made, based upon spreading earth delivered by wagons in an embankment built 5,000 miles away and under very different conditions.

It cannot be denied that there is a wide field of usefulness for detailed cost analyses. But their chief advantage accrues on the work or with the parties where they originate, and is found in the certainty with which they disclose weak points in organization, or extravagant methods of working. With the growing tendency toward the elimination of waste in all kinds of production, and with increasing competition among contractors, there is an excellent opportunity for the application of such a system. To make particular data of any very general value, however, there should be some such uniform system of accounting as has been applied so satisfactorily by the New England Water Works Association to the statistics of water works operation.

When estimating the probable cost of work, even by the best system of cost analysis which can be devised, there are yet so many indeterminate factors peculiar to each piece of work that the estimate of an engineer who has had extended experience in that particular line is much to be preferred to the most accomplished cost analyst's theoretical deductions.

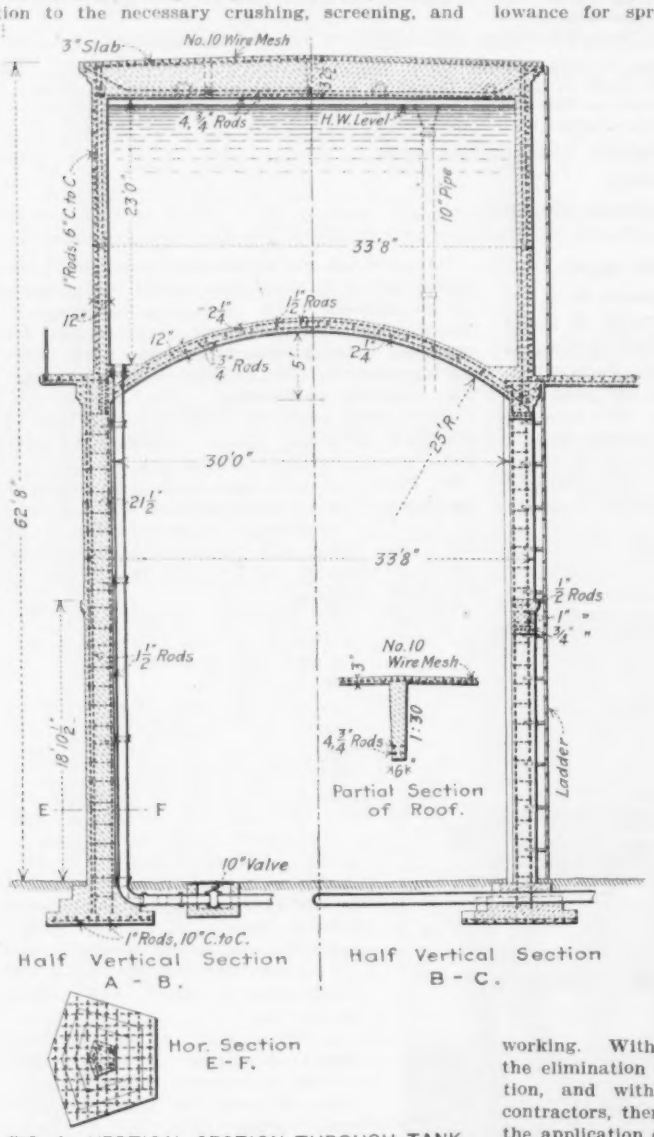


FIG. 2. VERTICAL SECTION THROUGH TANK.

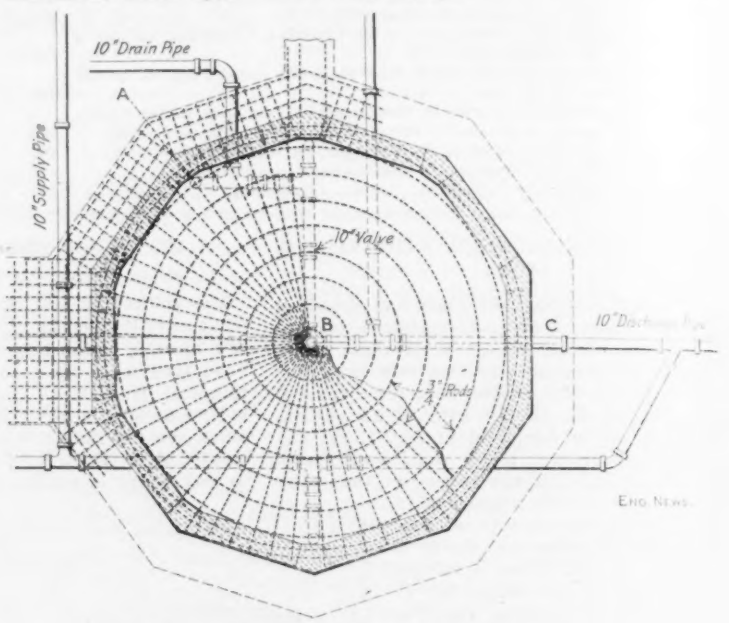


FIG. 3. REINFORCEMENT IN TANK BOTTOM.

THE SALE OF SEWAGE-FED OYSTERS will doubtless be prohibited in New York City hereafter, on the strength of the following addition to the Sanitary Code:

No. 185—No oysters shall be held, kept or offered for sale anywhere in the city of New York without a permit in writing from the Board of Health and subject to the rules and regulations of said board.

It is expected that the Board of Health will make sure that the oysters are produced under proper sanitary conditions before giving permits in accordance with this regulation.

OPERATIONS OF THE DAVYHULME SEWAGE WORKS, MANCHESTER, ENGLAND.

The report of the Rivers Department of the City of Manchester, England, for the year ending March 27, 1907, describes the operations of the several sewage purification works under the charge of that committee (John Frowde, Chairman, Town Hall, Manchester, England). The principal works treating the sewage of Manchester are located at Davyhulme. Here and in the vicinity the city now owns 214 acres of land, nearly half of which is occupied by works already built and the remainder of which is used for roads, embankments, etc., or reserved for future constructions.

The Davyhulme works consist first of screens and catch-pits designed to retain floating and heavy matter, respectively; open septic tanks preliminary to the treatment of sewage on contact beds; and sedimentation tanks preliminary to storm filter beds.

During the year under review, an average of 42,735,000 U. S. gals. per day were received at the works, which was slightly less than the average for the previous year. The average per capita daily flow ranged from a minimum of 66 U. S. gals. for the four weeks ending Sept. 12, 1906, to a maximum of 91 gals. for the four weeks ending Jan. 2, 1907. The greatest daily flow during the year (Oct. 18, 1906) was 122,630,000 U. S. gals. for the entire day, but from 7.15 to 8 a. m. the flow was at the rate of 208,000,000 U. S. gals. per 24 hrs. The average rainfall for the year (four stations) was 26 ins., as compared with 29 for the previous year. A measurable amount of rain fell at Davyhulme in 190 days.

The average daily quantity of sewage passed through the septic tanks during the year was 26,408,400 U. S. gals. The amount of sludge removed from the septic tanks averaged 21,400 lbs. per 1,000,000 U. S. gals. The average amount of sludge per 1,000,000 U. S. gals. for the past four years has been 18,700 lbs. In one of these years it averaged as low as 11,200 lbs. per 1,000,000 U. S. gals.

The sedimentation tanks dealt with an average of 16,326,000 U. S. gals. per day. The sludge removed from them averaged 28,420 lbs. per 1,000,000 U. S. gals.

Nearly all the sludge was taken to sea by a sludge steamer. This steamer made 163 trips with a total of 20,049 miles for Manchester, and 25 trips with a total of 3,200 miles for the adjoining municipality of Salford. Sludge presses were worked at intervals during the year, but they handled only a small quantity of sludge, half of which (800 long tons) was sold to neighboring farmers.

Of the total volume of sewage received at the works, 86.7% was passed through either the contact or the storm filter beds, and of that not passed through either kind of bed, practically all went through the sedimentation tanks. Of the sewage not filtered, only 0.75% of the total flow exceeded the limit of 5 parts of storm water to 1 part of dry weather flow. The primary contact beds (46 acres) treated septic tank effluent during the year at an average rate of 574,000 U. S. gals. per acre per day, including all resting periods.

A plant for washing the cinders composing the contact beds, designed by Mr. A. B. Ogden, Manager of the works, is described in the report as follows:

The filtering medium, as it is brought from the beds for washing or is delivered for making up the loss, is tipped on an elevated platform, approached by an inclined roadway. This platform is about 11 ft. above the ground level, on which the wagons receive the washed and graded medium ready for conveyance to the beds.

The medium is tipped into a small sump receiving water, and thence passed on to a jig, the first portion of which is constructed on sheet-iron plates, the latter portion consisting of a screen of ¼-in. mesh.

The material, as it passes over this jig, is thoroughly cleansed by means of fixed water jets. The water for this purpose is pumped from one of the sewage tanks into an elevated feed reservoir.

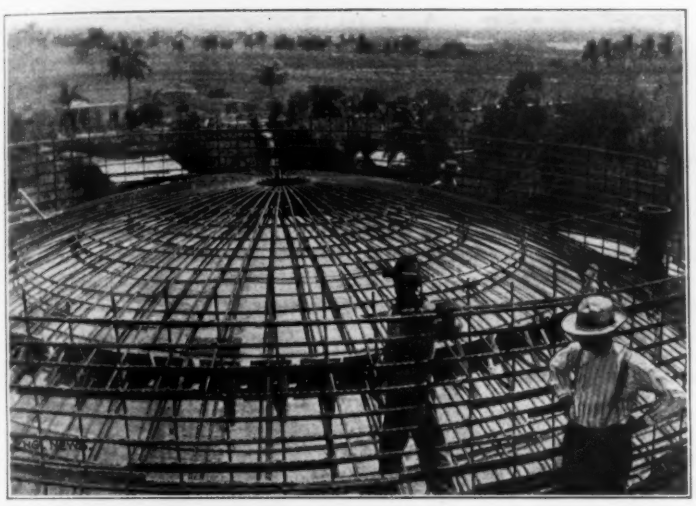


FIG. 4. PLACING REINFORCEMENT IN TANK BOTTOM.

The washed medium above ¼-in. size passes over an inclined fixed screen of 2-in. mesh; anything below this size is collected in wagons underneath, and the larger material passes into another wagon. [The "wagons" are dump cars, running on railway track.]

The material which passes through the ¼-in. mesh screen, consisting of fine cinder and slurry, falls on to a second inclined plane of corrugated iron, and then over a ⅛-in. mesh screen. The material retained on this screen is collected in wagons, and at present tipped, but is of a character suitable for the construction of filter beds where fine medium is required.

The slurry which passes away from the washer is first received into a wagon, which is allowed to overflow. By this means the fine cinder is retained in the wagon, and afterwards tipped. The lighter slurry is carried away with the water to settling tanks specially constructed to allow of the removal of the deposited matter without interfering with the working of the tanks. The slurry passes to the tank sludge channel, and at present is either sent to sea or pressed. The overflow water from the settling tanks passes to the sewage-carrier.

The plant is easily capable of dealing with 150 [long] tons of filtering medium per day, the amount of sewage used in cleansing operations being about 40,000 gals. [48,000 U. S. gals.]

The total cost of the plant was £350 [$1,500]. This cost includes preparation of site, construction of inclined and elevated roads, concrete retaining walls, boiler, engine, settling tanks, etc.

Speaking of the contact beds, the report states that "sufficient experience has now accumulated to enable accurate figures to be given as to the cost of maintenance and ultimate renewal of primary contact beds."

Accordingly, a summary is given to show the cost of operating a group of six contact beds from the time that the beds were started until the filtering material was taken out. These figures, together with the number of fillings per bed and the quantity of septic effluent treated by the six acres, are as follows:

WORKING COSTS OF PRIMARY CONTACT BEDS
(Beds 1-a to 10–6 acres.)

Average number of fillings per bed	2,690
Gallons (U. S.) of septic tank effluent dealt with by the 6 acres (12 beds)	4,613,624,000
Total maintenance cost	$4,087.26
Total renewal cost (40½ cts. per cu. yd.)	13,705.20
Maintenance cost per 1,000,000 U. S. gals. filtered	.89
Renewal cost per 1,000,000 U. S. gals. filtered	2.97
Actuating values per 1,000,000 U. S. gals. filtered	.25
Total working cost per 1,000,000 U. S. gals. filtered	$4.11

"Maintenance," as used above, includes all labor for "removing deposit from the surface of the beds, raking, forking, weeding, and attention where necessary to the drainage of the beds."

The item "actuating valves" is based on the total labor cost of this operation for the whole works. The "renewal cost" includes removing, washing and replacing the old cinders and making good the lost material; also, charges for the washing plant and the cost of "dealing with the slurry."

The original cost of the cinders used for filtering material "screened and delivered into the bed" was from 3s. to 3½s. (73 to 85 cts.) per cu. yd.

Several references have been made to contact beds. These are primary contact beds. At the time of the report only one secondary contact bed, ½-acre in area, had been built, but a loan of some $400,000 for constructing 27 acres of secondary beds on land immediately below the Davyhulme works had been authorized by the Local Government Board and the work of construction was in progress.

The report abounds with tables showing in detail the operations of the works during the year, the quantities dealt with, costs, and various analyses. The average per capita cost of sewage treatment for the past five years, including sludge disposal, filtration, all maintenance and renewals and all general expenses, but with certain receipts deducted, has been as follows:

1902-3, 6.56d., or 13.1 cts.
1903-4, 6.14d., or 12.3 cts.
1904-5, 5.95d., or 11.9 cts.
1905-6, 7.77d., or 15.5 cts.
1906-7, 9.07d., or 18.1 cts.

During the period the estimated population served by the works increased from 567,570 to 576,620.

Many of our readers will remember that Dr. Gilbert J. Fowler was for many years chemist of these works. He is now Consulting Engineer to the Rivers Department of Manchester.

ROAD DRAGS AND THE ILLINOIS ROAD DRAG LAW.

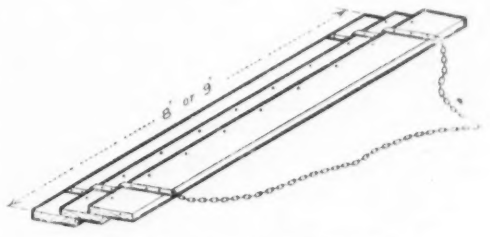
How to get passable roads at moderate cost is the great problem of all sections of the country. One of the simplest means of improving dirt roads in some sections of the country is the use of road drags to keep the roads reasonably smooth. In recognition of the value of these devices and to encourage their use the Illinois Legislature of 1907 passed an act authorizing local highway authorities to contract for road dragging at not over $1 per mile for each time dragged in the four months December to March, inclusive, and 75 cts. per mile during the remainder of the year. In letting such contracts preference is to be given to adjoining land owners or tenants. Not less than 20 ft. in width must be dragged, if conditions permit.

The law in question provides that the dragging shall be done "as nearly as practicable in accordance with the instructions of the highway" or other road commissioners. As a guide for all concerned the Illinois Highway Commission has issued "Bulletin 2, Road Drag Law and Rules for Dragging." The rules given "are based on the actual experience of users of road drags in various parts of the state, where they have been found perfectly practicable."

The bulletin states that "roads properly dragged will dry out weeks earlier in the spring" than those not dragged and will not rut "so readily during the winter." Dragging on an average of twice a month is sufficient, with more frequent dragging in winter and spring than during the summer.

Drags should be about 8 ft. long, of light rather than heavy woods. At least two round trips with an 8-ft. drag are required for a 20-ft. roadway. Flat or center-hollowed roadways should be crowned with a road machine before being dragged. Repeated proper use of a drag "will gradually crown a road," but a drag is designed to maintain and not to build a road.

Three kinds of drags are illustrated in the bulletin: one made from a log, split in half, a similar one, of plank, with a square timber as a trailer, and a lap-plank drag. The first two types (except that the second type had no trailer) were illustrated in our issue of June 14, 1906, in connection with a reprint of an earlier bulletin of the Illinois Highway Department. The lap-plank drag is shown herewith.



Lap-Plank Drag for Use on Dirt Roads.

Three paragraphs may be quoted from the bulletin, as follows:

Unless the road is in the right condition, the work of dragging will be wasted. One thing to be insisted upon is that the work be done at the right moment. The right time is when the road is wet. The muddier it is the better the results. On a road that is in extremely bad condition and the mud is very deep, it is probable that the form of drag shown here in Fig. 3, known as the lap-plank drag, can be worked to better advantage.

In the summer time and early fall, dragging should be done while it is actually raining, for unless the rain is exceptionally heavy and long continued, the water will penetrate the dry roadbed so fast that the surface will be comparatively dry when the drag is used after the rain has stopped, with the result that the road surface will work up in crumbs. When this happens it is a sign the road is too dry. The nearer it is possible to spread the mud over the road as a mortar, much in the same way a mason works mortar with a trowel, the greater the improvement produced.

Under no conditions should a road be dragged when it is dry. This merely crumbles up the surface and makes a layer of the loose material which quickly becomes dust and is again turned into mud which will hold water on the surface of the road; and this is exactly the condition that is to be prevented. Drag when the road is good and muddy. Don't drag when it is dry. Drag whenever possible and at all seasons of the year. If a road is dragged immediately before a cold spell, it will freeze in a smooth condition.

From the earlier bulletin it appears that "road drags have been known and used spasmodically for many years," but that "it remained for Mr. D. Ward King, of Missouri, to show by persistent demonstrations the wide application and practical importance of the road drag." Mr. A. N. Johnson, Springfield, Ill., is State Engineer of Illinois.

BROKEN STEEL TAPES may be quickly mended by a repair kit recently put on the market by the Chicago Steel Tape Co. A thin metal sleeve, made to fit all widths and thicknesses of tapes, is coated on the inside with a composition of solder and flux which melts at so low a temperature that it makes a complete adhesion with the broken ends of the tape when heated by an ordinary match. This repair can be made in the field in a minute or two without taking the temper out of the tape or obstructing its being coiled in the case. The splicing sleeves are put up in small round wooden boxes which hold several dozen sleeves and yet can be carried in a vest pocket.

PONTOON OR FLOATING DRAWBRIDGES.

The pontoon or floating type of drawspan for crossing navigable streams is an interesting but unusual type of structure which is only adopted under exceptional conditions, but has been used to a limited extent for both railway and highway bridges. The cost of construction and operation are small as compared with structures of the ordinary type, and those now in use in this country have given satisfactory service for a number of years. The most important of these are two spans in the bridge carrying the Chicago, Milwaukee & St. Paul Ry. over the Mississippi River between Prairie du Chien, Wis., and North McGregor, Ia. This was built in 1874 by Mr. John Lawler, Fellow Am. Soc. C. E. (as described below), who in 1883 described it in a paper read before the American Society of Civil Engineers.*

An early pontoon railway bridge was mentioned in the discussion of this paper by Mr. Don J. Whittemore, M. Am. Soc. C. E., Chief Engineer of the Chicago, Milwaukee & St. Paul Ry., who also suggested that it would be practicable to use two pontoons as a double-leaf bridge to give a clear opening of 1,000 ft., which could be opened or closed in four minutes. He stated that in 1851 Mr. Henry R. Campbell, Chief Engineer of the company then building the Vermont & Canada Ry. and Northern New York Ry., built a pontoon draw to connect these roads at the outlet of Lake Champlain near Rouse's Point, N. Y. A bridge had been proposed, but the application for permission to build this was defeated in the New York legislature, and Mr. Campbell therefore devised the floating draw, to be operated under the legislative acts governing the operation of steamboats on Lake Champlain. The pontoon was 300 ft. long, 30 ft. wide and 7 ft. deep, with adjustable aprons at the ends to connect with the trestle approaches. The variation in water level was about 7 ft., and there was very little current. This draw was operated by a dismantled locomotive and was in use until April, 1868.

As to the economic features of this type of bridge, we quote the following from a paper on "Mississippi River Bridges," by Mr. F. B. Maltby, in the "Journal" of the Western Society of Engineers for August, 1903:

> The use of this class of structure is, of course, limited to a very few locations on the river, as it can only be used where there is a gentle current, and where the ice can be taken care of without much trouble. I have not been able to ascertain the cost of this bridge, but it would seem that the first cost should not exceed 20% of that of an ordinary drawbridge at the same structures would have expected that it would have been replaced with a permanent structure of the ordinary character. The fact, however, that it is owned and operated by one of the most progressive roads in the west, that it has been maintained for 26 years, and that the third structure has been built at this place, would indicate the belief of the officials of the road that it is an economical permanent structure.

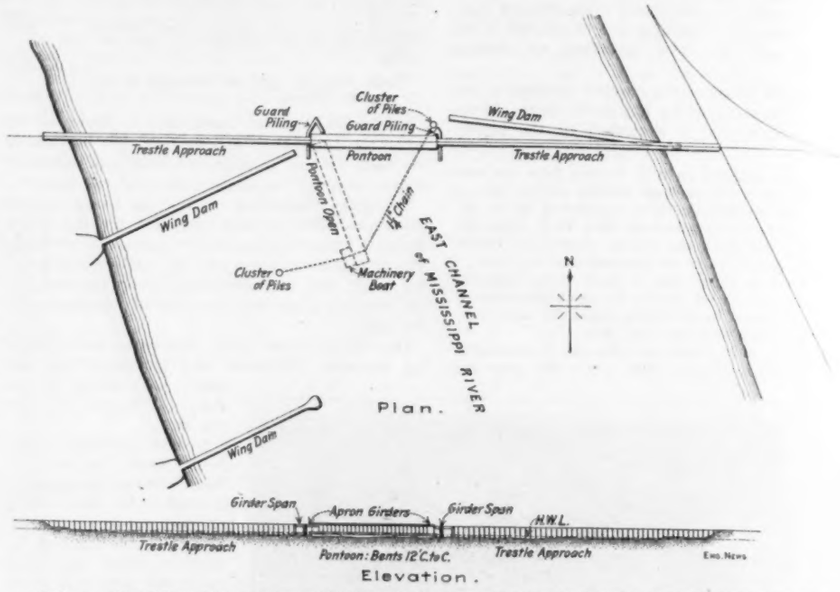


FIG. 1. GENERAL PLAN AND ELEVATION OF THE PONTOON AND APPROACHES AT THE PRAIRIE DU CHIEN END OF THE BRIDGE BETWEEN PRAIRIE DU CHIEN, WIS., AND NORTH McGREGOR, IA. CHICAGO, MILWAUKEE & ST. PAUL RY.

(A similar trestle, with pontoon, crosses the North McGregor channel at the other end of the bridge.)

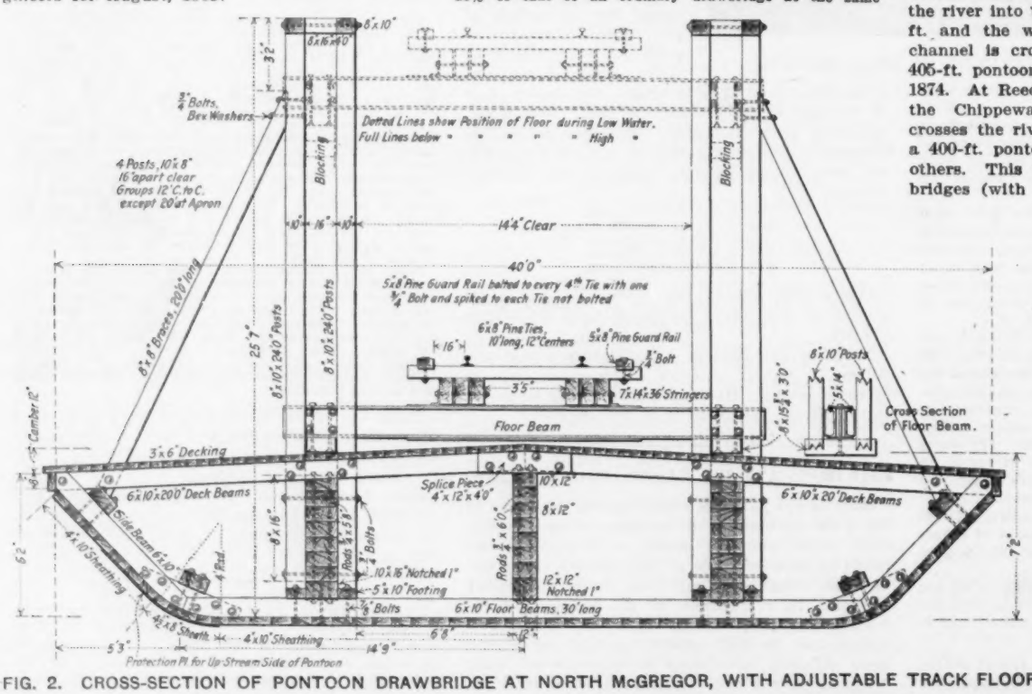


FIG. 2. CROSS-SECTION OF PONTOON DRAWBRIDGE AT NORTH McGREGOR, WITH ADJUSTABLE TRACK FLOOR.

We are led to believe from the description of the original structure (of the Prairie du Chien bridge. C., M. & St. P. Ry.), that the principal consideration in deciding upon its use was one of first cost, and one conversant with the fate of the ordinary wooden railway point. From the above history it is safe to say that the life of the structure is from 12 to 15 years, which is at least half as much as the ordinary so-called permanent structures built at the time of the original bridge. If the annual repairs amount to 5% of the first cost, the total cost, at the end of the 26 years of its life, is still considerably less than that of the ordinary structure. This is omitting interest charges, which at the time of building were very much more serious in amount than at present.

*Transactions; American Society of Civil Engineers, March, 1884.

RAILWAY BRIDGES WITH PONTOON DRAW SPANS: C., M. & ST. P. RY.

The Chicago, Milwaukee & St. Paul Ry. has the distinction of operating four pontoon draw spans. Of three crossing the Mississippi River, two have been in use for 33 years and another for 25 years. The fourth was built only about two years ago. At the point where the railway crosses the Mississippi River from Prairie du Chien, Wis., to McGregor, Ia., an island divides the river into two channels, the eastern one 2,000 ft. and the western one 1,500 ft. wide. Each channel is crossed by a timber trestle, with a 405-ft. pontoon draw. This bridge was built in 1874. At Reed's Landing, near Wabasha, Minn., the Chippewa Valley branch of the railway crosses the river by a trestle 3,500 ft. long with a 400-ft. pontoon draw of similar design to the others. This was built in 1882. Both of these bridges (with three pontoons) have been rebuilt since their first construction. The variation of level in the river is 22.3 ft. at North McGregor and 15 ft. at Wabasha. There is a swift current, with logs and ice at certain seasons. The fourth bridge of this type was built about two years ago to carry the White River Valley extension across the Missouri River at Chamberlain, S. Dak. Here the variation in water level in the river is 11.45 ft. and the current is less swift than in the Mississippi, but there is a great quantity of large drift to be passed.

The long continued use of the pontoon type of bridge on the Chicago, Milwaukee & St. Paul Ry. makes these structures of particular interest, and we give some description of them herewith. For much of the information we are indebted to Mr. C. F. Loweth, M. Am. Soc. C. E., Engineer and Superintendent of Bridges and Buildings for the railway company.

PRAIRIE DU CHIEN AND McGREGOR BRIDGE.—The length of the line at this crossing is about 1½ miles, and includes two river channels of 2,000 ft. and 1,500 ft., with an island

1,500 ft. wide. The approaches and the island crossing are partly embankments, but there are about 3,150 ft. of trestle and two Howe trusses of 114 ft. span. The two pontoons are 405 ft. long. The bridge was originally designed and built in 1874 by Mr. John Lawler, who had charge of the ferry transfer of passengers and freight for the railway company, the cars being formerly taken across on ferry barges. This type of draw was patented by Mr. Lawler, who described it in a paper read before the American Society of Civil Engineers, as already noted. He stated that the cost was about one-sixth of the estimated cost for an ordinary swing span (with foundations) at this point. Mr. Lawler died in 1891. The bridge was rebuilt in a more substantial manner in 1882. In 1898 the eastern end (at Prairie du Chien) was again rebuilt, and provided with a new pontoon; the western (McGregor) end was similarly improved in 1900. Fig. 1 is a plan and elevation of the Prairie du Chien channel crossing and pontoon draw.

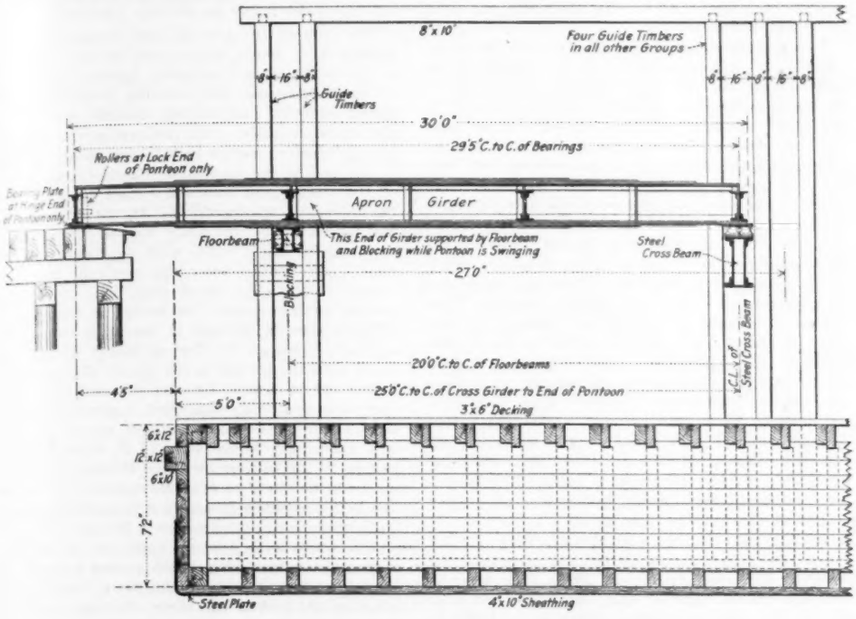


FIG. 3. APRON CONNECTION AT END OF PONTOON.

As originally built, each draw was a single pontoon or barge, 408 ft. long, 41 ft. wide on the deck and 30 ft. at the bottom, and having a depth of 6 ft. The framing and bottom timbers were of Norway pine, and the deck of white pine. Three longitudinal bulkheads, built up solid of horizontal timbers bolted together extended the full length, dividing the hull into four compartments. The accompanying illustrations represent the new pontoon of the McGregor draw, built in 1900 by the Bridge and Building Department of the railway company.

The pontoon, Fig. 2, is 405 ft. long over all, 6 ft. 5 ins. deep at the sides and 7 ft. 5 ins. at the center, the deck being crowned or cambered. Its width is 40 ft. on the deck and 26 ft. on the flat bottom, the bottom corners being rounded off to a radius of 4 ft. The deck, side and bottom beams are timbers 6 × 10 ins., forming frames spaced 2 ft. c. to c. To these the 4 × 10-in. sheathing planks and 3 × 6-in. deck planks are spiked. The sheathing at the bottom corner of the upstream side is protected by iron plating, as shown in the cross section. Three solid bulkheads of horizontal timbers are built between the floor and deck beams, and against the side bulkheads are bolted the 8 × 10-in. upright timbers 24 ft. long which form the guides for the floor beams. These timbers are seated on stringers on the bottom beams, and are in groups of four, spaced 16 ins. apart in the clear. They are connected at the tops by longitudinal timbers, with filler pieces between. These groups of timbers are spaced 17 ft. 4 ins. apart c. to c. transversely, and 12 ft. c. to c. longitudinally. Inclined braces extend from these timbers to the side beams of the hull.

The track floor beams are pairs of 15-in. steel channels, set 5 ins. apart back to back, with top and bottom cover plates. These are 20 ft. 6 ins. long, and their ends extend between the sets of vertical timbers. Under each end is bolted a timber shoe which rests upon the deck when the track is at its lowest position, or upon the blocking when it is raised. Packed stringers laid on these floor beams carry ties 6 × 8 ins., 10 ft. long; these are 12 ins. c. to c., and upon them are the track rails and guard timbers.

Portable hydraulic 15-ton jacks on the deck of the pontoon are used in adjusting the track to the proper level. The track is readily raised or lowered, as may be necessary, by raising the floor beams one at a time, and putting in or taking out the necessary blocking. The jacks are mounted on wheels so as to be readily placed, one at each end of the floor beams. The blocks used are 7 ins. thick, a lift or drop of 7 ins. being given at each movement of the jacks at each successive floorbeam. The whole length of track can be raised or lowered several inches by two men in a few minutes. The range of variation in level of the track on the pontoon is 14 ft.; when the river rises above the 14-ft. stage the approaches have to be raised. The maximum change of level of track at one movement is 18 ins., due to a rapid rise, or 10 ins. due to a rapid fall of the river. As the pontoon is depressed or given an excess draft of about 7 ins. by the weight of the train, the track level is adjusted to be a few inches above that of the approaches before the train passes upon it. Guard piers of piling protect the lock end of the bridge and also the stern of the pontoon when swung open; the latter pier is in triangular form, to divert logs, drift and ice to either side of the pontoon. While the length of the pontoon is 405 ft., the clear waterway when open is about 365 ft., the 40 ft. width of the pontoon lying inside the opening.

At 25 ft. from each end of the pontoon is a box girder floor-beam to which is pivoted an apron 30 ft. long, having main and transverse framing. Near the outer end is another floorbeam of the ordinary construction. This arrangement is shown in Fig. 3. At the pivoted end of the draw, the outer end of the apron slides upon and is supported by a plate on the abutment or end of the fixed structure when the bridge is closed. At the swinging end of the bridge, the outer end of the apron has rollers which engage with the seat upon the abutment. While the pontoon is swinging, the end cross beam of this apron is supported by blocking, as shown. The outer cross beam carrying the girders of the apron is held in position by a guide and buffer device which allows for lateral movement as the bridge closes. Rubber-faced rollers ride against the sides of the vertical guide timbers, and the roller shaft is carried in a casting which has a movement in guides parallel with the floor beam. Heavy coiled springs are fitted between this casting and a bracket on the beam, so as to return it to position if shifted laterally. This is shown in Fig. 4. The two 114-ft. Howe truss spans already mentioned are the end spans of the fixed approaches on either side of the North McGregor draw span. In order to give an easy grade connecting with the pontoon, each of these truss spans is made with a double deck, the upper deck being adjustable as to grade and level. The movable floor system is very similar to that of the pontoon, and is operated by jacks and blocking in the same way as that of the latter. At the Prairie du Chien draw span of the same bridge there is a girder span at each end of the approach (Fig. 1). Apron girders connect the track on the pontoon with that on the approaches, as in Fig. 3. While the pontoon is swinging, the girder is supported near the end of the pontoon by a steel floor beam with blocking as required. The apron at the hinged end of the pontoon has its shore end sliding upon a bearing plate on the abutment. The apron at the swinging or lock end of the pontoon is fitted with rollers which ride upon the abutment.

The pivot around which the pontoon swings consists of a large pile, securely driven, and having the upper part turned to receive an iron sleeve of 12-in. wrought iron pipe. On the down stream side of the pontoon, and about 7 ft. from the end, are bolted plates, as shown in Fig. 5, which project beyond the hull, and have in the outer edges a semi-circular slot to engage the face of the pivot pile. Around the back of the pile is

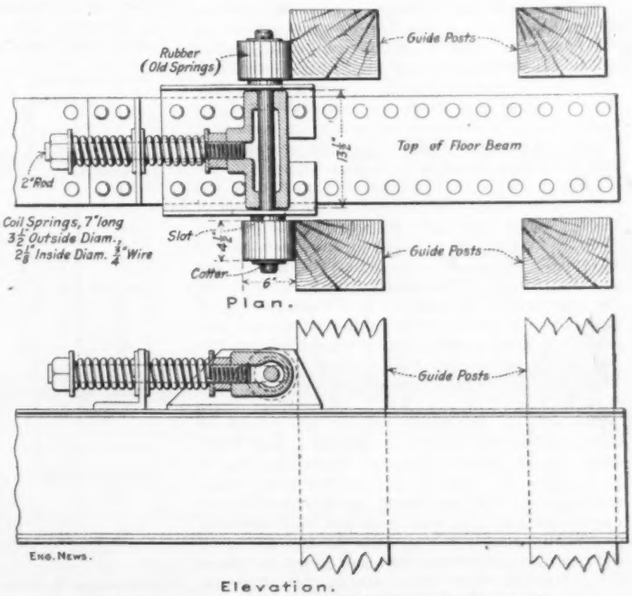


FIG. 4. GUIDES FOR END FLOORBEAM OF APRON.

passed a hinge strap or collar, 1¼ × 6 ins., the ends of which have forged lugs with holes for 2½-in. pins passing through the collar and plates. Cotters prevent the pins from rising. Adjacent to the hinge plates is a block 10 × 10 ins., bolted to the pontoon and having fenders or striking timbers bolted to its face. This engages with bumper piles as the pontoon closes. Originally

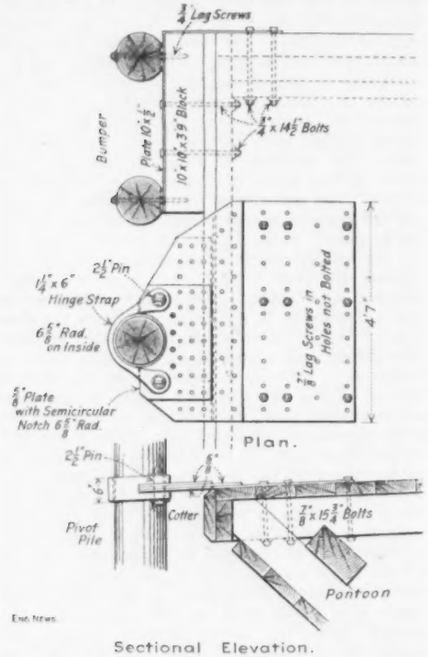


Fig. 5. Pivot with Connections for Pontoon Draw Span at North McGregor, Ia., C., M. & St. P. Ry.

the steel plates above mentioned projected far enough beyond the hull to have a 12-in. hole through which the pivot pile passed. This, however, was liable to bind and become distorted due to the vertical movement of the pontoon. The new arrangement, which was first used at the Wabasha bridge, allows of greater freedom and is more easily repaired or replaced.

When the pontoon is closed, it is held in position by a T-head on an iron bar or shaft projecting from the side of the deck at the upstream corner of the swinging end. As the bridge closes, this head (in a vertical position) passes between a pair of piles; the shaft is then revolved by a lever, bringing the head or lock to a horizontal position so that it takes a bearing against iron plates on square upright timbers secured to the backs of the piles. Rollers on the side of the pontoon bear against iron plates on the faces of these piles so as to prevent wear as the pontoon rises and falls. A nut on the threaded end of the shaft allows of adjustment to secure the proper position of the pontoon when locked. At the fixed end of the pontoon is a V-shaped lug or projection, placed centrally between the rails, and engaging with a socket on a head-block at the end of the approach; this centers the pontoon, and holds it in exact position, so as to keep the rails in line.

The rails at the fixed end are lap-spliced or scarfed along the middle of the head, and fitted with curved guard pieces, as shown in Fig. 6. The splices are held in a cast-iron shoe, made with an open end, and when the bridge is closed the bridge and approach rails are held together firmly and closely. The rails at the swinging end are lap-spliced, but do not clamp in shoes as at the fixed end. The rails on the approach at the swinging end of the draw, are left unspiked for a distance of 12 or 15 ft.; they are connected by tie-bars, and a 300-lb. counterweight tends to pull them in a down-stream direction. As the draw closes, rollers carried by arms bolted to the rails ride upon track plates on the approach ties, and the projecting ends of the rails engage with the loose ends of the approach rails, crowding the latter back into line. The lap splice is then held firm by the pull from the counterweight. No trouble has been experienced with rail connections due to rails creeping. The rails on the approaches have slip or expansion joints made of switch rails, about 50 ft. back from the opening, inside guard rails being spiked to the ties at this point. The rails on the pontoon are anchored, so that they will not creep in either direction.

The operating machinery is mounted on a separate float or pontoon 30 × 56 ft., placed at the downstream side of the swinging end, as shown in the general plan, Fig. 1. This is secured to the main pontoon by three timber beams having iron shoes fitted to pins in brackets on the two pontoons. These allow for independent vertical motion of the smaller float, but hold it in position laterally, one of the beams being parallel with and the others at right angles to the main pontoon. On the float is a 25-HP. hoisting engine, over the drum of which is passed a slack chain which lies across the bed of the river and is anchored at each side of the opening. The bridge is opened in about one minute by the force of the current, and is closed by the engine in about three minutes. Four men are required during the open or navigation season; and three men during the closed season, when logs and ice have to be passed through. The North McGregor pontoon was opened 1,176 times during the open season of 243 days in 1906, which is equivalent to 4.84 movements daily. The greatest number of movements during any seven-day period of that season was 60.

REED'S LANDING BRIDGE.—This bridge (near Wabasha) was built by the railway company in 1882, and rebuilt in 1891. The river is about 2,400 ft. wide at the low water slope at this point, and the bridge consists of 2,300 ft. of pile trestle (with one Howe truss span of 114 ft.) and a 400-ft. pontoon draw giving a clear opening of about 350 ft. Here large quantities of logs have to be passed in the spring, these being made up into rafts at a point below the bridge.

The pontoon is 396 ft. long and 41 ft. wide. It is 6 ft. 6 ins. deep for 316 ft., but for 20 ft. at each end the depth is 7 ft. 6 ins., the change being made in a distance of 20 ft. The purpose of this is to give greater buoyancy to check the increased submersion as the weight of the train comes upon the end. The pontoon was built of Douglas fir, from Washington. It was rebuilt during 1907, creosoted timber being used for the bulkheads and sheathing in the parts most liable to decay. Its floor and side timbers are 6 × 10 ins., forming frames 24 ins. apart, to which are spiked the 4-in. sheathing and 3 x 10 in. deck planks. There are three longitudinal bulkheads as in the pontoon already described. The vertical guide timbers are also similarly arranged, but the floor beams are timbers 10 × 12 ins., 22 ft. long. These are 10 ft. 6 ins. apart, and upon them are packed timber stringers carrying the ties. The last floor beam at each end is 30 ft. from the end of the pontoon. From this extend timber stringers, the outer ends of which rest upon the abutments when the bridge is closed. These prevent the load of the train from being carried directly upon the ends of the pontoon. These stringers and the floor beams are adjusted by hydraulic jacks as already described.

The rails at the pivoted end of the draw have square butt joints with those on the approach. A short guard rail is bolted against the outside of each pontoon rail, having projecting ends flared outward and bearing against the sides of the approach rails. The pontoon is also centered by a projecting bar or lock which enters a socket in the end of the approach; the downstream side of the socket and lock are curved to conform to the swing of the pontoon. At the outer end of the pontoon, the rails are halved for about 10 ins., making scarf joints with the fixed rails. The fixed ends project over the pontoon, and have rollers at the side which bear upon track plate on the pontoon while the bridge is swinging.

The bridge is pivoted in the way already described, although at first a heavy U-bolt was used, embracing the pivot pile. The bridge is locked when closed by the tee-head or "hammer" device already described. Against the downstream side of the main pontoon, at its outer end, is a machinery barge 46 × 24 ft. 5 ft. deep, attached to the former by three timbers pivoted at the ends to allow of free movement. A locomotive boiler supplies steam to a two-cylinder hoisting engine with cylinders 10 × 15 ins. The main pinion gears with a spur wheel on the shaft of a sprocket wheel over which passes a chain anchored at either end of the draw opening and lying on the bed of the river. It leads along the deck, and guide wheels cause it to run vertically to and from the drum. It is not wrapped around the drum, but simply passes over it. The chain links are of 1¼-in. iron.

CHAMBERLAIN BRIDGE.—The pontoon of the bridge across the Missouri River at Chamberlain, So. Dak., is of similar construction to that at Prairie du Chien, and is operated in the same way. The length of the pontoon is 360 ft.

HIGHWAY BRIDGES WITH PONTOON DRAW SPANS.

LACON, ILL.—An interesting case in which the pontoon type of draw span has been used in a highway bridge is the bridge crossing the Illinois River at Lacon, Ill. The plans and specifications

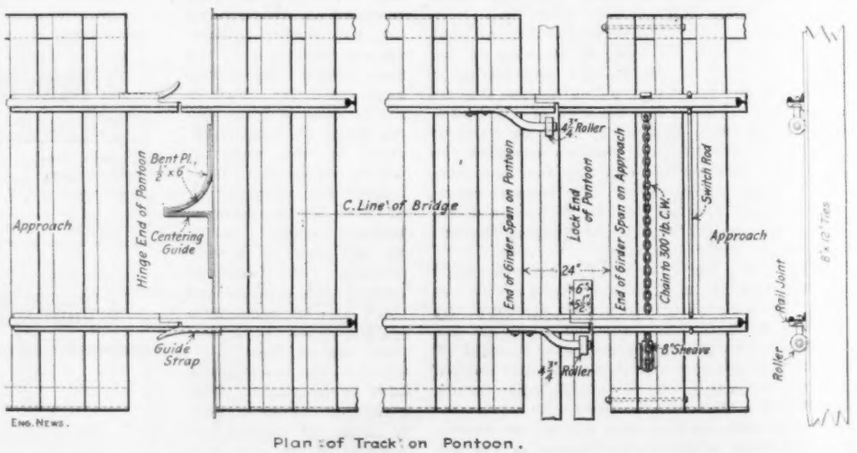


FIG. 6. RAIL CONNECTIONS AT SWINGING END, AND CENTERING LOCK AT HINGED END OF PONTOON.

for this were prepared in 1879 by the late Geo. F. Wightman, C. E., of Lacon, and the bridge was built in 1889 under his direction, by Mr. John Lawler, of Prairie du Chien, Wis. The bridge is 1,000 ft. long, with pile trestle approaches of 500 ft. on the east and 300 ft. on the west. The width of roadway is 16 ft. The length of the pontoon is 200 ft., with a width of 24 ft. and depth of 6 ft. Its draft is about 12 ins. The

river has a depth of about 38 ft. at low water, and has a maximum rise of 18 ft. above this level. The roadway for 40 ft. at the end of each approach and 40 ft. at each end of the pontoon is movable. The inner end of each movable section is hinged, while the outer end has a bolster to which long vertical screws are attached. On the vertical timbers between which the moving ends slide, are castings upon which are seated

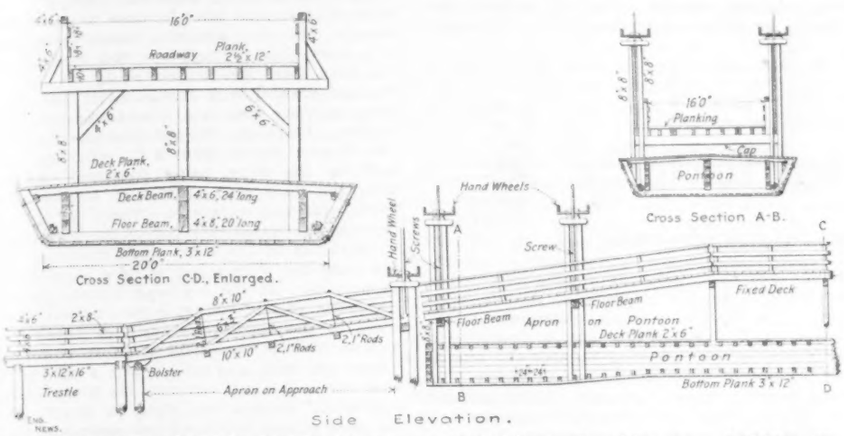


FIG. 7. HIGHWAY PONTOON BRIDGE ON THE ILLINOIS RIVER AT LACON, ILL. (APRONS ADJUSTED TO LEVEL BY SCREWS.)

nuts fitted with handles, and the bridge men adjust the level as required. This is shown in Fig. 7. The draw is hinged on a pile at one corner, and is opened and closed by a steam winch and chain. The new pontoon built in 1905 was 250 ft. long. It had beams of white oak, with bottom and side planking of Washington fir secured by boat spikes 5/16 × 7 ins. The joints of deck and planking were well calked. All other lumber was white pine, except that oak was used for the floors and posts.

The original cost of the bridge was about $17,000. It has been in constant use except for about four weeks in 1905, when the new pontoon was built. The maintenance expense is very light. The bridge is operated by two men; a day man at $55 and a night man at $45 per month. It is a toll bridge, owned by the city, and the receipts are from $3,500 to $4,000 per year.

CHICAGO, ILL.—Temporary pontoon draw spans have been used in three or four cases during the reconstruction of bridges over the Chicago River. In 1902, when the old swing bridge at State St., was removed to make way for a bascule bridge, the contractors proposed to operate a temporary foot-bridge with a 117-ft. draw span of timber trusses pivoted at one end and carried by a pontoon at the other end. This was not built, however, but in 1905 a similar system was adopted for temporary bridges at three river crossings. These bridges carried wagons and electric cars as well as foot passengers. The one at 22nd St. had a 100-ft. draw span, pivoted at one corner, and having the free end supported on a pontoon. Screw jacks provided for variations of water level in the river, and the bridge was operated by an electric hoist with a slack chain laid across the river bed. These structures were described and illustrated in our issues of Aug. 21, 1902, and Dec. 28, 1905.

NORTHWICH, ENGLAND.—A novel type of pontoon draw span was built in 1900 to cross the River Weaver at Northwich, England. A steel truss span of 112 ft. is supported near one end on a pontoon or floating chamber, the span thus having unequal arms and giving a clear waterway of 54 ft. The object of the pontoon in this case, however, is not to float the bridge, but simply to relieve the weight on the roller path on account of unstable foundations. This interesting structure was described and illustrated in our issue of Sept. 20, 1900.

PONTOON BRIDGES IN INDIA.

In all the pontoon bridges thus far described, only the draw-span has been carried by the pontoons. In several parts of the world, however, bridges have been built in which the entire structure consists of short spans supported by pontoons or boats, forming what was called a "bridge of boats." Modern highway bridges of this kind over rivers in India have been described in a paper on "Pontoon Bridges for Road Traffic over Rivers in Bengal" presented to the Institution of Civil Engineers (England) by Mr. Edward G. Barton, M. Inst. C. E. The bridges are five in number, ranging in length from 310 ft. to 558 ft. The rivers crossed flow through the flat alluvial sandy soil of the plains into the Ganges River, and periodically overflow their banks. Their courses are tortuous and are constantly changing (as the banks are eroded and cave), and their beds are liable to be scoured out or silted up. In most cases permanent bridges would be too expensive, on account of the depth of foundations necessary and the requirements for training works to maintain the channel in permanent position. Under these conditions, the pontoon type of bridge was adopted, and the bridges were built in 1900-02. Their general construction is shown in Fig. 8.

The pontoons are spaced 31 ft. c. to c., and carry two lines of 10-in. I-beams. The beams are 29 ft. 6 ins. long, connected by smaller transverse beams at the ends and middle of each span. The transverse floor planks rest on the 10-in. beams, and are held in place by the wheel-guard timbers, which are bolted to the beams. At each side are curved standards of steel tees, seated in brackets bolted to the webs of the beams. These carry three lines of galvanized iron wire and a top rail consisting of a steel angle. These fences are connected across the pontoons by chains as shown. The roadway is 8 ft. wide and the width between standards is 11 ft. The spaces in the roadway between adjacent spans are filled in with timbers resting on the lower flanges of the end cross-beams, as at A, in Fig. 9. Where the bridge is not level, however, or where there are expansion roller paths, a convex iron grating is used; this has one end pivoted on one span, while the other end slides on the adjacent span. This is shown in Fig. 10. After the annual storms and floods it is necessary to adjust the slope of the banks where the pontoons rest upon them, so that the grade will not exceed 10% in any span. On steep grades, the floor is covered with 2 or 3 ins. of earth to facilitate the passage of bullock wagons. The roadway is designed to carry a live load of 3.66 tons and a dead load of 4.63 tons, or a total distributed load of 8.29 tons.

As the rivers are navigable for boats for about half the year, each bridge has a draw-span. This is not opened by floating (as in the pontoon bridges previously described) but is a hinged span. On the end of the adjacent fixed span is a gallows frame with sheaves on the top. A chain attached to the top of each post is led through a pulley block hooked to the floor system of the draw span, and back to the sheave, from which it drops vertically to a drum on the post of the frame. Each drum has worm gearing operated by a crank handle fitted to the end of the worm shaft. An automatic brake sustains the load at any point. Two men can raise the span in 15 minutes and lower it in 6 minutes.

The following is an extract from Mr. Barbor's paper, relating to some interesting features of the bridges:

The pontoons are cylinders 6 ft. diameter and 30 ft. long, with conical ends, making the total length of the pontoon 36 ft. 8 ins. They are built of 3/16-in. steel plates with internal stiffeners, and each is provided with a manhole. The chief reasons for adopting this form of pontoon were: 1. In the dry season the pontoons are liable to rest on uneven ground either in the bed of the river or on the slope of its banks, and this form gives the greatest strength to resist the stresses to which they are then subjected. 2. It is water-tight when wholly immersed. 3. The single pontoon is a cheaper form of construction than double pontoons of smaller size. Each pontoon is subject to a dead weight of 8.98 tons and a live load of 3.66 tons, or a total load of

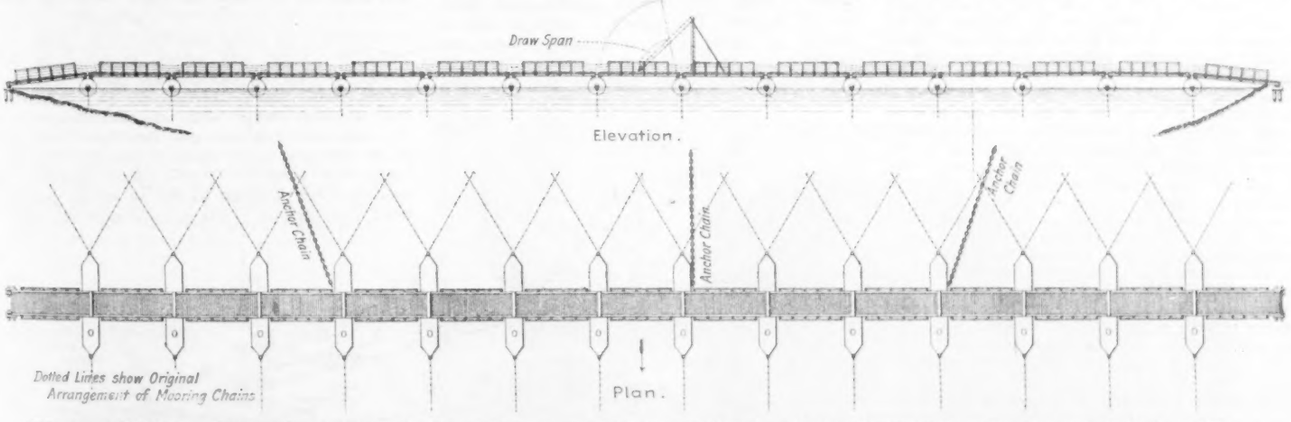


FIG. 8. PONTOON BRIDGE WITH LIFTING DRAWSPAN, FOR HIGHWAY TRAFFIC IN THE DARBHANGA DISTRICT OF BENGAL, INDIA.

12.64 tons, so that when two adjacent spans are loaded the pontoon between them is half immersed. The extra weight to be carried by the pontoon on which the lifting gear is operated (due to the weight of the lifting-gear and of the span lifted) is 4.21 tons, so that the maximum weight which this pontoon has to carry is 13.19 tons. When the draw-span is raised the pontoon in this case actually sinks 6 ins. to 9 ins. over the half-immersion line. In order to increase the stability of this pontoon an extra pontoon (although not absolutely necessary) has been placed alongside it.

The pontoons were originally moored to mushroom anchors, three to each pontoon, by means of ⅛-in. chains, two up-stream 100 ft. in length and one down-stream 70 ft. in length. The positions of these chains are indicated by dotted lines in Fig. 8. They were fastened to rings at the ends of the pontoon which are attached to an iron rod running through the centre of the pontoon longitudinally. This rod is in two pieces, joined in the center by a turnbuckle. It was found, however that, owing to the great scour and silting up of the rivers, and also to the trees and debris brought down in flood-time, considerable trouble and anxiety for the safety of the bridges was caused by anchors dragging; the pontoons were tilted and pulled down by the weight of the trees and debris on the anchor-chains, and sometimes these chains were broken. The first alteration made was to fasten the anchor-chains to the roadway instead of to the ends of the pontoons, with the result that no more tilting of the pontoons was experienced. The arrangement finally adopted, however, was to moor the whole bridge by means of three ⅝-in. anchor-chains in place of the ⅛-in. chains from each pontoon. One chain, 200 ft. long, is fixed to the center of the bridge and attached to a screw-anchor in mid-stream. Two chains, 300 ft. long, one on each side of the center (half way between the center and the shore-ends) are attached to shore-anchors on the banks, on the up-stream side of the bridge.

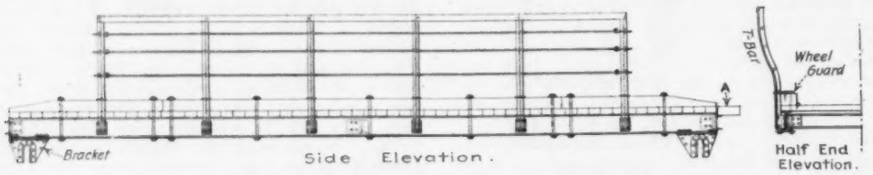


FIG. 9. ONE OF THE FIXED SPANS FOR PONTOON BRIDGE.

In order to provide for variation in length of the bridge, due to the rise and fall of the river, or to its resting wholly or partially on the uneven bed of the river, the ends of the gangways in certain places are supported on rollers free to roll on roller-paths mounted on the pontoons. Five or more sets of those rollers are provided to each bridge; one on each end, one on each sloping bank and one at the opening span. Each of these roller-paths (Fig. 10) allows for an alteration in length of about 12 ins..

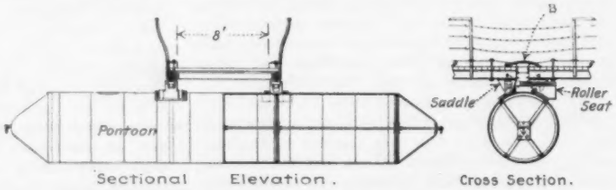


FIG. 10. PONTOON FOR PONTOON BRIDGE IN INDIA.

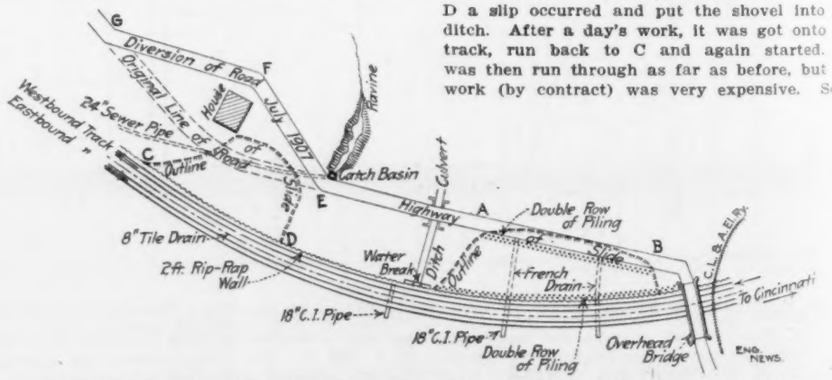
At each end of the I-beams of each span is bolted a bracket resting on a steel bar 3 ins. diameter, fixed in a saddle which sits in a bracket riveted to the pontoon. Any tendency of the saddle to slide up out of the bracket is prevented by a stud-bolt with a ring head. Only one end of the span revolves on the bar, the other end is keyed by means of a pin and cotter. The brackets originally used were of cast iron, but it was found that the jar and vibration of heavy loads broke them. Wrought-iron brackets were therefore substituted, and have proved quite satisfactory. Some of the cast-iron saddles also broke, and as there is considerable strain on these where they are keyed, they are now being gradually replaced by wrought-iron saddles.

The erection of these bridges involved no riveting at the site, and should it ever be required to reduce or increase the length of a bridge or to transfer it to some other place, this can be done easily and quickly. Most of the pontoons are accessible for painting in the dry season. Access to the undersides of those which are always afloat is obtained by raising the pontoon till it is just clear of the water by means of four levers, one on each side of the pontoon and on each side of the roadway. Each lever has about 2½ tons to lift. When the water is too deep for this method to be adopted, two barges are floated on opposite sides of the pontoon, and the ends of the girders are jacked up till they come out of the saddles. The pontoon, being then free, is allowed to turn bottom upwards, which it does owing to the weight of the cast-iron brackets riveted on top of it. Neither of these operations necessitates stoppage of the traffic.

THE IMPROVEMENT OF A SLIDING CUT ON THE CLEVELAND, CINCINNATI, CHICAGO & ST. LOUIS RY.

In the description of the reconstruction of the Chicago Division of the Cleveland, Cincinnati, Chicago & St. Louis Ry., in our issue of May 16, 1907, particulars were given of a troublesome wet and sliding sidehill cut, and we have recently obtained some information as to the remedial measures adopted. This is of particular interest, as difficulties of this kind are not infrequently met with, and there is little information published as to the methods of dealing with them. The matter of prime importance in all such cases is thorough drainage, to provide for the escape of water flowing through the soil or the removal of water imprisoned in pockets. In many cases, however, the main idea is to hold the sliding mass, and drainage measures are regarded as a secondary consideration.

The railway has at this point a double track on a curve of 2½° (with 7 ins. elevation) at the foot of a cut about 40 ft. deep in sloping or sidehill ground. The material is a wet yellow clay, and lies on a stratum of shale or hardpan. Water soaking through the clay mass gives a smooth and slippery or lubricated surface to the shale, and upon this the clay slides. Near the top of the cut is a public highway known as the Homestead Road, and at one point there is a house on the hill side. The accompanying cut shows a plan of the work at this cut. In January, 1906, considerable trouble was experienced between the points A and B, and a number of teams were employed to keep the road passable by filling in from the top the earth that was settling away towards the cut. Between these points the road was about 75 ft. out of line, and it seemed probable that the railway company might have to buy land to rebuild the road on a new location. Night watchmen had to be employed to warn teamsters of the breaks in the road.



PLAN OF SLIDING CUT NEAR HOMESTEAD, O.; CLEVELAND, CINCINNATI, CHICAGO & ST. LOUIS RY.

In the spring, two rows of piles were driven along the lower side of the original line of the road and two rows of piles were also driven at the toe of the slope. The rows were 4 ft. apart, and the piles also 4 ft. apart, the rows being staggered. These lines of piling are shown by heavy dots on the plan. The piles were driven with a 2,800-lb. hammer, and were driven as far as possible into the substratum of shale, the penetration being generally about 6 ft. A layer of brush was filled between the two upper rows of piles, and this was covered with earth to a level about 2 ft. below that of the road. The road was then back-filled to the original line and covered with broken stone. No brush filling or other work was done at the lower row of piles. There are over 900 piles in these rows. Two French drains about 3 ft. wide were then built up the slope, being excavated into the shale and filled with about 4 ft. of loose rock. These were very expensive. There has been no apparent movement of either of the double rows of piling, or of the slide, since this work was done, and the work seems to have accomplished its purpose. At the overhead bridge, however, the lower row of piling (which is here unassisted by the upper row) has been forced over, with an average movement of 2½ ft. at the top.

Later on, there was trouble with a slide at the west end of the cut, and near the house already mentioned. During one night in March, 1907, the mass slipped and threw the westbound track against the eastbound track, covering it with about 8 ft. of mud and throwing the eastbound track about 3 ft. out of line. The place had been under suspicion for some time and watchmen had been stationed, so that no train accidents resulted. A steam shovel was run through the foot of the slide, working from C to D, but while this improved the conditions as far as the track was concerned, it made them worse for the highway. In the summer of 1906, a catch-basin had been built at the ravine shown on the plan, and a long line of 24-in. sewer pipe laid to carry the water flowing from the ravine away from the cut. This worked fairly well until the slide of March, 1907, took place, when the pipe became disconnected and caused further trouble by leading the water into the slide. During the summer of 1907 there was continual trouble with the road passing in front of the house, and after the necessary negotiations the line of the road was changed and thrown behind the house, as shown at E, F. G. This appears to have settled the highway permanently.

There has been no trouble with the house, as its foundations extend through the upper soil and about 18 ins. into the shale. The last slide extended to the corner of the house, and the front yard went down about 5 ft. In December, 1907, the slide was again so close to the track as to be dangerous, and a steam shovel was again started at C. When it had worked about 75% of the distance to D a slip occurred and put the shovel into the ditch. After a day's work, it was got onto the track, run back to C and again started. It was then run through as far as before, but the work (by contract) was very expensive. Some of the material was then taken out by hand and with a railway ditching machine, forming a slope of 1 on 1¼. The face is still sliding at the rate of about 5 ins. per day, but no more trouble is expected, as the material will prob-

ably settle to its angle of repose before it reaches the track.

At the foot of a drain leading down the slope is a "water break" or concrete baffle wall to prevent the water (during the wet season) from spouting upon the track instead of flowing into the ditch. The ditch has been made of large capacity, and along it a rip-rap wall 2 ft. high has been built in each direction from the "water break" or baffle. This wall is just outside the ends of the ties and prevents washing of the roadbed by the water in the ditch. Three cross-drains of 18-in. cast-iron pipe extend under the roadbed from this ditch, discharging on the downhill side. There is also a line of 8-in. soft drain tile laid between the tracks, and having the proper outlets. The ditch capacity required

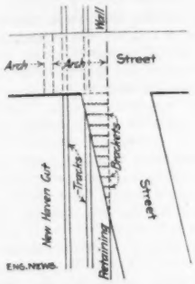


Fig. 1. General Layout of Skew Street Intersection.

by the drainage area involved is equivalent to a 7S-in. pipe, but since January, 1907, there has been no overflow upon the track.

The work thus far done has cost nearly $25,000, and although both tracks were ballasted with 14 ins. of gravel last spring, it is expected that the cost of maintenance per foot will be excessive, owing to the wet and unstable conditions. The methods above described were adopted after investigation by the engineers, but it is thought that all of the water from the ravine has not been intercepted by the catch-basin and drain, otherwise the second slide would not have occurred. It is not expected, however, that further slides will now develop at this point.

TESTS OF OLD BRIDGE METAL to determine whether any deterioration due to long service could be noticed have been made on several recent occasions by the German railway systems. This has been done by taking tension test specimens from different parts of the webs and flanges of girders taken from bridges removed after long service. The specimens from the end of a flange, which had been subjected to very much lower stresses than those from the center, would therefore represent more nearly the original metal as concerns absence of deterioration due to stress, and thus some estimate of the service deterioration could be formed by comparing the results of tests from center and end of flange, etc. A recent set of tests was made by the Königsberg division of the Prussian State Railways on metal from bridges which had been in service for 47 years on a well-traveled line. The results, reported in the "Zentralblatt der Bauverwaltung," and transformed by us into American measure, are as follows:

Source of Specimen.	Ult. tensile strength, lbs./sq. in.	Elongation, per cent.
Upper flange angle of floorbeam:		
End of beam	47,000	15.0
Center of beam	46,700	11.7
Web-plate of floorbeam, at mid depth:		
End of beam	45,400	11.5
Center of beam	44,100	7.5
Lower flange angle of floorbeam:		
End of beam	46,700	16.5
Center of beam	49,000	16.5
Upper flange angle of stringer:		
End of beam	48,800	18.3
Center of beam	49,200	20.0
Web-plate of stringer, at mid depth:		
End of beam	43,200	8.5
Center of beam	41,600	6.5
Lower flange angle of stringer:		
End of beam	48,100	11.0
Center of beam	45,200	11.0

These results indicate no recognizable difference between the specimens most and least subjected to changes of stress in service, and like some preceding tests of the same kind made in the last few years by the German railway administrations they indicate that no deterioration in physical properties was produced in service.

REINFORCED-CONCRETE BRACKETS UNDER SKEW SIDEWALKS.

By C. L. SLOCUM,* Assoc. M. Am. Soc. C. E.

In the improvements of the New York, New Haven & Hartford R. R. Co. at New Haven, Conn., in two instances, a new and improved highway which crosses the depressed tracks on reinforced-concrete arches is intersected just at the crossing by another street, the sidewalks on which overhang the tracks in the cut and are carried by brackets of reinforced concrete, projecting from the abutment faces, their under sides nearly conforming to the intrados curve of the neighboring arches, toward which, as shown diagramatically in Fig. 1, the cantilever portion extends until the last bracket is 14 ft. 6 ins. long or half the span of an adjoining full span arch.

Essentially the reinforcement consists of two railway rails bent to the shape or curve of the outer surfaces and laced together by 4 × ¼-in. bent plates and ½-in. bolts; a similar construction to that described in the general account of the bridges in this section in Engineering News, April 11, 1907, p. 392. A sufficient length of rail extends into the abutment or retaining wall to develop the full holding capacity of the bracket. This fabricated frame is practically enclosed in a 12-in. rib of concrete, which has some supporting powers of its own, the rib and abutment construction being monolithic (Fig. 2). These long skews are so situated that the sidewalks only have to be carried on the brackets, the roadway being over the solid fill.

The form work of this fabricated construction was comparatively difficult and tedious, but the results obtained seem to have amply justified the painstaking. The timber falsework trusses of the neighboring arches formed the intrados of the brackets, box forms of carefully matched boards between the brackets supporting and molding the slab construction. The concrete work was done last winter at low temperature, some of it when the thermometer registered about 20° F. above zero. It was executed in good time, notwithstanding the fact that the aggregate had to be dried out and freed from ice, and the water steamed for mixing. The mixing was expedited and the concrete quickly puddled into place. As soon as portions of the work were completed, heavy canvas was thrown over it, on top of which 2 or 3 ins. of warm sand were spread, and manure thickly spread over this. Boards over all prevented the protection from being disturbed. The laying of the granolithic paving was a subsequent operation, that was conveniently suited to milder weather. On account of the nature of the construction, the forms could not be removed when the concrete was green, so that the finishing of the surfaces was accomplished by carborundum blocks, when the forms were finally removed. The results obtained here rather prove that just as satisfactory work can be produced in cold weather as under normal and milder conditions.

This particular construction is a small part of a variety of extensive bridge work in connection with this improvement, in which the writer was in charge of the design under the supervision of Mr. W. H. Moore, M. Am. Soc. C. E., Engineer of Bridges.

*Assistant Engineer, N. Y., N. H. & H. R. R., New Haven, Conn.

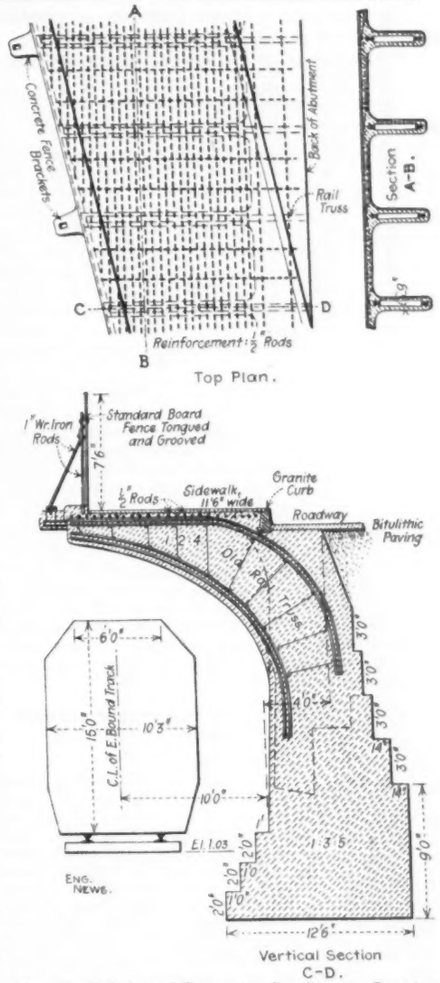


Fig. 2. Reinforced-Concrete Cantilevers Carrying Sidewalk, New Haven Cut, N. Y., N. H. & H. R. R.

FIG 3. SKEW BRACKETS AT GRAND AVENUE, NEW HAVEN.

THE THEORY OF LATTICED COLUMNS, A COMPARISON OF CANTILEVER BRIDGE CHORDS, AND A DISCUSSION OF THE QUEBEC BRIDGE SPECIFICATIONS.*

[Appendices 16, 17, 18 and 19.]

(With inset sheet.)

After studying the records of all previous tests of large columns, and then itself recording tests of two columns several times larger than any before tested, as noted in our preceding issues, the Quebec Bridge Commission entered into a field still more difficult and unexplored: the theory of column lattice. Appendix 16, in which the discussion of this subject is presented, is reprinted in full herewith. It forms a highly valuable contribution to the literature of column theory, none the less valuable because it is incomplete.

There is less pronounced novelty, but no less importance, perhaps, in the Commission's critical review of the Quebec Bridge specifications, Appendix 18, which we also reproduce in full. These specifications copied, and wholly without individuality, are only an expression of what the Commission criticized specifically in earlier parts of its Report: the absence of competent engineering direction of the enterprise. But the high unit-stresses which are also criticized do not owe their origin to this cause, since they were fixed by Mr. Cooper long after the bridge was first designed. The Commission seems to say that, apart from all errors of design, these unit-stresses reduced the margin of safety more than warrantable.

A Discussion of the Theory of Built-up Compression Members.

[Appendix 16.]

This discussion will be confined to columns of which the cross-section is rectangular in outline and which are built up of two or more parallel webs with stiffening angles, connected by lattice bars, tie plates, diaphragms, etc. In such columns the parallel webs carry the load and the connections serve a subsidiary purpose. For convenience the webs, considered apart from their connections, will be termed the web system and the connections the lattice system. In many bridges the continuous cover plates of the top chord belong to both the web system and the lattice system, inasmuch as they both carry load and serve as connections for the side plates.

In the design of the cross-section the arrangement and dimensions of the web system are first considered. Column formulas based on experiment are used for this purpose. These formulas give the average unit-stresses under which columns fail in terms of their length and radius of gyration. This radius is taken in the plane in which the column will probably fail by buckling or bending. A factor of safety is used in the design and a suitable arrangement of the cross-section of the web system adopted. The web system, in short, is designed from column formulas or the plotted results of experiments from which these formulas are deduced.

The design of the lattice system is quite a different matter. As a rule it depends on the judgment of the engineer guided solely by experience. He finds little or nothing in scientific text-books or periodicals to assist his judgment. Some lattice formulas are in existence but they are not generally known and their utility is more or less doubtful owing to the uncertainty of the data and assumptions on which they are founded. The unsatisfactory nature of the column formulas upon which the web system is designed is a matter of common knowledge among engineers, but the column formulas may be considered to represent exact science in comparison with the lattice formulas.

The lattice system performs two distinct functions. In the case in which each web of the web system carried its share of the load, that is to say when there is no transfer of load in any part of the column from one web to another, the lattice system simply acts as a side support to the web system and by means of it a long web is divided up into a number of short columns. The stresses thrown into the latticing by this action cannot be computed. In this case the load on the column is parallel to the axis but not necessarily coincident with it, and the curvature is assumed to be negligible. When, however, the load is inclined to the axis of the column, the lattice system has a different function. The angle of inclination may vary from point to point along the column owing to the curvature of the column. This curvature may be due to original bends or to the action of the load or to both combined. If the curvature is sufficiently small the variation of inclination due to it will be negligible. There remains, however, the original inclination or obliquity which is due to the method of application of the loads at the ends of the column. If the eccentricity of application is the same at each end and in the same plane with the axis of the column, there will be no obliquity other than that arising from the curvature of the webs which may be negligible. If, however, the eccentricities at the opposite ends are different or in different directions the obliquity may be of considerable amount.

If the curvature of the column be negligible the obliquity arising from the eccentricity will be the same at every point. This obliquity causes a transfer of load throughout the whole length of the column from one web to another. This transfer of load is accompanied by longitudinal shearing stresses in the lattice system. The obliquity also causes transverse shearing stresses at every cross-section of the column.

If the lattice system is considered to be sufficiently stiff the longitudinal shearing forces can be derived from the transverse shearing forces by the ordinary processes of statics as applied to elastic solids, and from them the lattice stresses and the lattice cross-sections may be computed.

If θ is the angle between the direction of the column axis and that of the load, S the transverse shear and P the load.

$$S = P \sin \theta$$

and since in practical cases θ is small this may be written

$$S = P\,\theta$$

if θ be expressed in radians or as the ratio of the total eccentricity to the length of the column.

This formula holds true also if the curvature of the column is great enough to require consideration. In such a case θ varies along the column, and in computations the column should be divided by cross-sections so close together that the difference in θ at two neighboring cross-sections may be disregarded.

So far the problem is comparatively easy. With the next step difficulties begin. The question now is, what value of the obliquity shall be chosen for design.

Since the obliquity depends upon inequality in the eccentricities at the ends, the maximum difference must be decided upon for design. It would seem reasonable to assume for this purpose equal eccentricities in opposite directions, so that if e be the assumed eccentricity at one end the maximum value of the obliquity will be given by

$$\theta = \frac{2e}{l}$$

l being the length of the column. Against this view, however, it may be urged that the chances of the maximum value to be assumed for e depends upon the excel- that therefore some smaller value should be chosen.

Evidently the strength of this objection depends upon the value assumed for the eccentricity. The safe maximum value to be assumed for e depends upon the excellence of the design both of the column and of the splices, on the accuracy of workmanship, and on the care and precision employed in the erection.

It is impossible to estimate with accuracy the value of e under any set of conditions, but reasonable limits for its values will doubtless be learned from experience and study. With bad work, and more especially bad fitting and weak splices at butt joints, the value of e may be much greater than it need be under other conditions of construction. In design, however, good workmanship and strong splices should be assumed. Theoretically the cross-sections of the latticing should be designed so that with the assumed eccentricity the latticing and the web systems will get their ultimate safe stresses simultaneously. This connection will be satisfied if the unit-stress in the latticing has the same factor of safety as the maximum compressive stress in the web system corresponding to the eccentricity.

Let P be the safe load;
A the area of the cross-section;

$$p = \frac{P}{A};$$

d the greatest diameter of the cross-section in the plane of the latticing;
r its radius of gyration parallel to d;
q the unit stress at the most compressed edge;
e the eccentricity of the load P;
then

$$q = p\left(1 + \frac{e\,d}{2\,r^2}\right)$$

[which equation is generally true only within the limit of elasticity], and consequently

$$e = \frac{2\,r^2}{d}\,\frac{q-p}{p};$$

$$\theta = \frac{2\,e}{l} = \frac{2}{l}\,\frac{2\,r^2}{d}\,\frac{q-p}{p};$$

$$S = P\,\theta = p\,A\,\frac{2}{l}\,\frac{2\,r^2}{d}\,\frac{q-p}{p} = A\,\frac{2}{l}\,\frac{2\,r^2}{d}\,(q-p)$$

In design all the quantities in the above expressions for e, θ, and S are fixed without difficulty with the exception of q, the extreme unit stress in the web system. It is evident from the formula for S that S becomes zero when $q = p$. Now the maximum value of q for which the formula

$$q = p\left(1 + \frac{e\,d}{2\,r^2}\right)$$

holds is, in general, the elastic limit. Consequently as p approaches the elastic limit, S approaches zero. Evidently, when p is equal to the elastic limit the load must be central and without obliquity and no part of it can be transferred from one web to another without inducing stresses in the second web in excess of the elastic limit.

The function of the latticing in such a case is simply to stiffen the webs and, as has already been said, the accompanying lattice stresses cannot be computed. The condition necessary for a theoretical computation of the stresses in the latticing is that the difference between p and q be reasonably large. Lattice formulas, of course, fix a value of q in a fashion, but only tests and experience can determine whether or not these give economical and safe results. Direct tests are difficult to apply and unless great care is exercised incorrect inferences may be drawn from them. A lattice column placed in a testing machine may fail in the web system, but this is no indication that the lattice system is strong enough for service in a similar column when in use as a bridge member. It may be that the obliquity of the load was too small to develop the lattice strength, and considerably less than the obliquity that would be found to exist on the bridge. With a greater obliquity the column might fail through the lattice system under a much smaller load. In other words, the failure of the webs is an indication that the full strength of the column has been nearly developed. The failure of the latticing may not be such an indication. The full strength of the column can be developed only by axial loading, and under such loading comparatively weak latticing may serve to develop this strength.

The full strength of the latticing can be developed only by oblique loading. The column strength in this case must be less than under axial loading, as otherwise failure would occur first in the web system.

The failure of lower chord A 9 L Quebec Bridge is an example of an insufficient lattice system. The webs bent and the lattices failed under a load only three-fourths of the specified maximum working load.

Determination of the area of a lattice bar cross-section.

The bar must be designed to take equal stresses in tension and compression. Let
P' represent the lattice stress;
A' the tension section;
A'' the compression section;
q' the unit stress in tension;
q'' that in compression.

The unit stress q'' must be computed by a column formula.

Now $P' = k\,S$, where k is a coefficient which can be calculated from the known arrangement and dimensions of the lattice and web system. This calculation will be taken up later.

Thus

$$P' = A'\,q' = A''\,q'' = k\,S$$

therefore

$$A' = \frac{k\,S}{q'} = k\,A\,\frac{2}{l}\,\frac{2\,r^2}{d}\,\frac{q-p}{q'}$$

$$A'' = \frac{k\,S}{q''} = k\,A\,\frac{2}{l}\,\frac{2\,r^2}{d}\,\frac{q-p}{q''}$$

The values q' and q'' in design should have at least the same factor of safety as q.

It may be more convenient in many cases to make A' represent the shearing area of the rivets and q' the shearing unit stress. The net area of the lattice bar has been selected in this discussion because in the Quebec Bridge it was weaker than the rivet areas, the lattice bar section being 1.15 sq. ins. and the rivet area 1.80 sq. ins. (3 rivets).

In the arrangement of the lattice system the free portions of the webs should have a value of $\frac{l}{r}$ less than that of the column as a whole.

Lattice Formulas.

In the foregoing discussion it has been shown that there are two points with regard to which there must be more or less doubt and in which no aid can be expected from theory; first, the stresses to which the latticing is exposed when the load is axial and, second, the value to be assumed for the maximum unit-stress in the web system.

The assumption is made that a satisfactory solution of the second difficulty is sufficient to provide for the

*This concludes our series of abstracts of the report of the Quebec Bridge Commission of Enquiry. For the preceding sections, see Engineering News: March 12, 1908, p. 288, Formal Findings; March 19, 1908, pp. 307-315, Abstracts of Report and of Appendices 3 to 11, incl.; April 9, 1908, pp. 404-405, Appendix 13 (Summary of Tests of Large Columns); April 16, 1908, pp. 421-423, Appendices 12 and 14 (The Wreck of the Quebec Bridge and the Stresses in the Bridge); April 23, 1908, pp. 453-459, Appendix 15 (Tests of Two Compression-Chord Models: The Largest Column Tests Ever Made).

first. All practical lattice formulas determine, in effect, the value to be assigned to q. The same factor of safety is used for q' and q'' as for q.

Mr. C. C. Schneider, Consulting Engineer, has called the attention of the Commission to an article on this subject by Professor Prandtl of Goettingen in the "Zeitschrift des Vereins Deutscher Ingenieure" of Dec. 28, 1907. Professor Prandtl assumes that the equation

$$q = p\left(1 + \frac{e\,d}{2\,r^2}\right)$$

holds up to the point of failure of the outer web. He makes q the ultimate strength of the portion of the outer web between neighboring lattice points. He necessarily makes q' and q'' in the formula for lattice bars represent ultimate strengths. He also discusses the allowance to be made in the value of r on account of want of stiffness in the latticing. In other respects his discussion corresponds to ours.

In the same journal a letter appears from Professor Engesser of Karlsruhe who also refers to the diminution of r owing to want of stiffness of the latticing. This letter does not contain sufficient information to enable a reader to follow the line of thought. However, in the formula for lattice bar areas, he seems to replace $\frac{q-p}{q'}$ and $\frac{q-p}{q''}$ by $\frac{q-p}{q}$ in which q is the ultimate strength of the material and p is the ultimate strength of the column which he determines by use of the Tetmajer formula. He also replaces the factor 2 by π. He states that he published his investigations in 1891 and 1893.

Mr. H. S. Prichard gives a rule which he had used for several years which makes $S = .015\ P$, in other words $\theta = .015$.

Mr. Szlapka, in his evidence, states that after a most painstaking search the only information he could find on the subject of lattice computations was that given in Johnson's "Modern Framed Structures." The experience of the Commission is practically the same as that of Mr. Szlapka for, except the rule in "Modern Framed Structures," all the information we have been able to find has appeared in the periodical press since the collapse of the Quebec Bridge.

Mr. Bindon Stoney, one of the earlier authorities on bridge construction, has given a method of computing statically the stresses in lattice existing in a bent column.

The article in "Modern Framed Structures" is as follows:

> There are no rules other than empirical ones in use by which the size and spacing of lattice bars for compression members are determined. * * * It has been suggested that, as our compression formulas all assume a certain extreme-fiber stress due to the flexure of the strut, from this known extreme-fiber stress we find an equivalent uniform load acting in the plane of the latticing which will produce this fiber stress and from this load find the stress in the lattice bars.

This method is equivalent to assuming that $q = f$ and $\theta = \frac{4\,e}{l}$; f being taken from the formula $p = f - c\,\frac{l}{r}$ or the formula

$$p = \frac{f}{1 + \frac{1}{c'}\left(\frac{l}{r}\right)^2}$$

when applied to the working stresses. The value of S thus becomes

$$S = P\,\theta = P\,\frac{4\,e}{l} = P\,\frac{8\,r^2}{l\,d}\,\frac{f-p}{p}$$

From the straight-line formula

$$\frac{f-p}{p} = \frac{c}{p}\,\frac{l}{r} = \frac{A}{P}\,\frac{c\,l}{r},$$

so that

$$S = \frac{P}{l\,d}\,\frac{8\,r^2}{}\,\frac{A\,c\,l}{P\,r} = \frac{8\,A\,c\,r}{d}$$

From the Rankine formula

$$\frac{f-p}{p} = \frac{1}{c'}\left(\frac{l}{r}\right)^2$$

so that

$$S = P\,\theta = \frac{8\,l}{c'\,d}\,P$$

The equations give very different values for S, even though the constants of the Rankine formula be computed so that the curve represented by it and the straight line represented by the straight-line formula are tangent at the point corresponding to the value of $\frac{l}{r}$ in question.

Mr. Szlapka used the rule in "Modern Framed Structures." He selected Rankine's formula and gave c' its value for square bearings. He, however, modified the method by using a central load instead of a distributed load. This modification had the effect of making the area of the lattice bar one-half of that given by the method suggested in "Modern Framed Structures." Mr. Szlapka finally adopted a larger cross-section than his method gave, and one which, in his judgment, was sufficient.

If he had tested the method fully he would have found it capable of giving areas ranging up to ten times the area computed by him, a result which would have shown the unreliability of this method. He might, of course, have come to the conclusion that a rule capable of giving such different results was valueless.

In an article in "Engineering," Sept. 27, 1907, Professor Keelhoff of Ghent University states that in 1893 he developed a lattice formula which has been more or less extensively used. When thrown into the form of the theoretical formula previously given, the following results are obtained: Professor Keelhoff multiplies the expression for θ by the coefficient $\frac{\pi}{2}$. Thus, instead of $\theta = \frac{2\,e}{l}$, his method gives $\theta = \frac{\pi\,e}{2}$, this change being the result of a theoretical study in which he adopted Euler's sinusoid curve as the probable form of a bent column. He also replaces q by f taken from the column formula for working stresses

$$p = f - c\,\frac{l}{r}$$

It is thus apparent that the methods of lattice computation which have been used in practice, when thrown into the form adopted in this discussion, simply assign values to the unknown q, and in some cases multiply the theoretical obliquity by the factor 2 or $\frac{\pi}{2}$.

The following is a résumé of the lattice formulas discussed in this appendix:

Theoretical Formulas.

$$c = \frac{2\,r^2}{d}\,\frac{q-p}{p}$$

$$\theta = \frac{2\,e}{l} = \frac{2}{l}\,\frac{2\,r^2}{d}\,\frac{q-p}{p}$$

$$S = P\,\theta = p\,A\,\frac{2}{l}\,\frac{2\,r^2}{d}\,\frac{q-p}{p}$$

$$= A\,\frac{2}{l}\,\frac{2\,r^2}{d}\,(q-p)$$

$$A' = \frac{k\,S}{q'} = k\,A\,\frac{2}{l}\,\frac{2\,r^2}{d}\,\frac{q-p}{q'}$$

$$A'' = \frac{k\,S}{q''} = k\,A\,\frac{2}{l}\,\frac{2\,r^2}{d}\,\frac{q-p}{q''}$$

q to be determined by judgment and not to exceed the elastic limit.

q' and q'' to have the same factor of safety as q.

Formulas Used in Practice.

Prandtl — makes q the ultimate strength of the portion of the outer web between neighboring lattice points and q' and q'' the ultimate strength of the lattice bars used.

Engesser — replaces $\frac{q-p}{q'}$ and $\frac{q-p}{q''}$ by $\frac{q-p}{q}$ respectively, using ultimate values. He also multiplies θ by $\frac{\pi}{2}$, i. e. makes $\theta = \frac{\pi\,e}{l}$

Prichard — makes θ constant $= .015$.

"Modern Framed Structures" — makes $q = f$ of the working stress formula

$$p = f - c\,\frac{l}{r} \text{ or } p = \frac{f}{1 + \frac{1}{c'}\left(\frac{l}{r}\right)^2}$$

and also multiplies θ by 2, i. e. makes $\theta = \frac{4\,e}{l}$

Szlapka — modified the rule in "Modern Framed Structures" by not using the multiplier 2 in the value of θ, i. e., made $\theta = \frac{e}{l}$ and also used the formula

$$p = \frac{f}{1 + \frac{1}{c'}\left(\frac{l}{r}\right)^2}$$

giving c' its largest value, viz: 36,000.

Keelhoff — makes $q = f$ of the working stress formula $p = f - c\,\frac{l}{r}$ and also multiplies θ by $\frac{\pi}{2}$.

Computation of k in formula $P' = k\,S$.

The method of making this computation will be illustrated by a numerical example.

For this purpose lower chord A 9 L Quebec Bridge will be taken.

On the assumption that the latticing is sufficiently stiff to enable the webs to act as a unit, the relation between the longitudinal shear S' in the length of one panel of latticing and the transverse shear S at the end of the panel is given by the statical formula $S' = \frac{S\,X\,Q}{I}$, where X is the length of one lattice panel, Q the moment of area about the central axis of the chord cross-section perpendicular to the lattice planes, of that portion of the web cross-section which lies outside the given plane of longitudinal shear, and I the moment of inertia of the whole chord cross-section about the same axis.

Evidently maximum S' corresponds to maximum Q which, in chord A 9 L occurs between the center webs. The numerical values are $Q = 6,439$, $X = 72.75$, $I = 362,640$, dimensions being given in inches.

Therefore $S' = 1.55\ S$ between the center webs. Similarly between the outside web and the center web $Q = 5,313$, giving $S' = 1.28\ S$.

In a lattice panel there are four bars arranged two and two as the diagonals of a square of which the side is 54.36 ins., this being the distance between the axes of the outside webs.

Therefore

$$P' = \frac{S'}{4} \times \sqrt{2} = .358\,S',$$

$P' = .35 \times 1.55\ S = .54\ S$ between center webs, and

$P' = .35 \times 1.28\ S = .45\ S$ between outside and center web.

Thus the values of k are .54 and .45.

From the design of the chord it is evident that the net area of the lattice bar is governed by $k = .54$, while the rivets connecting the bar to the outside web are determined by $k = .45$ and those connecting the bar with the inside web by the difference of these values which is .09. That is to say, if 5 rivets were necessary to connect the bar to the outer web, only 1 would be required for the connection with the inner web.

Transverse shears and bending moment exist in the webs due to the transverse shear S on the cross-section of the chord.

The maximum transverse shear in the outer web occurs in the space between the quadrilateral panels of latticing. It is equal to $\frac{1}{2} \times \frac{54.36}{72.75} \times 1.28\ S = \frac{1}{2} \times .747 \times 1.28\ S = .48\ S$, if the small bending moments in the webs due to the assumed distribution of stress at the plane of section be neglected; the maximum shearing stress on a center web section is thus $.02\ S$, the sum being $.50\ S$, half the shearing stress on the cross-section of the chord.

Thus 96% of the transverse shear is carried by the outer webs and only 4% by the inner webs in the space between the panels.

The difficulty of determining theoretically the values to be assigned to the quantities $\frac{q-p}{q'}$ and $\frac{q-p}{q''}$ in the formulas

$$A' = k\,A\,\frac{2}{l}\,\frac{2\,r^2}{d}\,\frac{q-p}{q'},$$

and

$$A'' = k\,A\,\frac{2}{l}\,\frac{2\,r^2}{d}\,\frac{q-p}{q''},$$

has been pointed out. It will be of interest to compare the solutions of this problem as given by the various methods that have been described in a numerical example and also to compare the corresponding lattice cross-sections.

For this purpose the web system of lower chord A 9 L, Quebec Bridge, will be selected. It will be sufficient for the present purpose to consider the formula for the tension section A'.

In this chord, $l = 684$ ins., $r = 19.7$ ins., $d = 67.5$ ins., $A = 780$ sq. ins., $k = .54$. Thus,

$$A' = .54 \times 780 \times \frac{2}{684} \times \frac{2 \times 19.7^2}{67.5} \times \frac{q-p}{q'} = 14\,\frac{q-p}{q'} \text{ sq. ins.}$$

Prandtl: $\frac{l}{r}$ for outer web between lattice points $= 44$;

ultimate strength of outer web, say, $48,000 - 210 \times 44 = 38,760$.

Specified unit load on column $p = 24,000$;

Tensile strength of lattice bar $q' = 60,000$ $\therefore \frac{q-p}{q'}$

$$= \frac{38,760 - 24,000}{60,000} = \frac{14,760}{60,000} = .25;$$

$$A' = 14 \times .25 = 3.50 \text{ sq. ins.}$$

If the unit load for the column had been determined by the formula

$$p = 16{,}000 - 70\,\frac{l}{r} = 16{,}000 - 70 \times 34.7 = 13{,}571,$$

we should have had

$$\frac{q-p}{q'} = \frac{38{,}760 - 13{,}571}{60{,}000} = \frac{25{,}189}{60{,}000} = .42$$

and $A' = 104 \times .42 = 5.88$ sq. ins.

Engesser: $q = q' = 60{,}000$;

$$p = 48{,}000 - 210\,\frac{l}{r} = 48{,}000 - 210 \times 34.7 = 40{,}713$$

$$\frac{q-p}{q'} = \frac{60{,}000 - 40{,}713}{60{,}000} = \frac{19{,}287}{60{,}000} = .32;$$

$$A' = \frac{\pi}{2} \times 14 \times .32 = 7.04 \text{ sq. ins.}$$

When this formula is used, the cross-section of the latticing does not vary with the load.

Prichard:

$\theta = .015$. Now theoretically $\theta = \frac{2}{l}\,\frac{2\,r^2}{d}\,\frac{q-p}{p}$

$$= \frac{2}{684} \times \frac{2 \times 19.7^2}{67.5} \times \frac{q-p}{p}$$

$$= .0333\,\frac{q-p}{p};$$

$$\therefore \frac{q-p}{p} = \frac{.0150}{.0333} = .45;$$

$$\therefore \frac{q-p}{q'} = \frac{q-p}{p}\,\frac{p}{q'} = .45\,\frac{p}{q'} = \frac{.45 \times 24{,}000}{40{,}000} = .27.$$

Assuming that 40,000 in tension represents the same factor of safety as 24,000 (in the case of this column) in compression,

$$A' = 14\,\frac{q-p}{q'} = 14 \times .27 = 3.78 \text{ sq. ins.}$$

Since $\frac{p}{q'}$ is constant for all factors of safety this result applies to all loads.

"Modern Framed Structures":

(1) Straight line formula.

$q = 24{,}000$, $p = 24{,}000 - 105 \times 34.7 = 20{,}356$
$q' = 30{,}000$

$$\therefore \frac{q-p}{q'} = \frac{24{,}000 - 20{,}356}{30{,}000} = \frac{3{,}644}{30{,}000} = .12;$$

$$A' = 2 \times 14 \times \frac{q-p}{q'} = 28 \times .12 = 3.36 \text{ sq. ins.}$$

In this formula evidently $\frac{q-p}{q'}$ is constant for all factors of safety and therefore for all loads.

(2) Rankine's formula.

$$\frac{q-p}{q'} = \frac{f-p}{q'} = \frac{\frac{p}{c'}\left(\frac{l}{r}\right)^2}{q'}$$

Assuming $p = 24{,}000$, $c' = 18{,}000$, $\frac{l}{r} = 34.7$, $q' = 30{,}000$, $\frac{q-p}{q'} = .0535$;

$$A' = 2 \times 14 \times \frac{q-p}{q'} = 28 \times .0535 = 1.50 \text{ sq. ins.}$$

If c be made 36,000 the values are

$$\frac{q-p}{q'} = .0267 \quad A' = .75 \text{ sq. ins.}$$

Evidently the same results will be given by all factors of safety, i. e., for all loads.

Mr. Szlapka's method, if he had used the proper value of k, would have given $A' = .37$ sq. ins. He assumed that the panels of latticing were square, whereas they were oblong and the lattice bars were not diagonals.

In the actual design, however, he made $A' = 1.15$ sq. ins.

Keelhoff:

$$\frac{q-p}{q'} = \frac{f-p}{q'} = \frac{24{,}000 - 105 \times 34.7}{30{,}000}$$

$$= \frac{24{,}000 - 20{,}356}{30{,}000} = \frac{3{,}644}{30{,}000} = .12;$$

$$A' = \frac{\pi}{2} \times 14 \times \frac{q-p}{q'} = 1.57 \times 14 \times .12 = 2.64 \text{ sq. ins.}$$

The value of $\frac{q-p}{q'}$ will not be altered by using different factors of safety for p, q and q', and therefore applies to all loads.

These results, arranged for compression, are collected in the accompanying table.

Author.	$\frac{q-p}{q'}$	A' sq. ins.	
Prandtl	.25	3.50	for $p = 24{,}000$.
	.42	5.88	for $p = 13{,}571$ from formula $p = 16{,}000 - 70\,\frac{l}{r}$.
Engesser	.32	7.04	for all values of p.
Prichard	.27	3.78	" " "
"Modern Framed Structures"	.12	3.36	for all values of p, straight formula.
do.	.0535	1.50	for all values of p, Rankine's formula, $c' = 18{,}000$.
do.	.0267	.75	for all values of p, Rankine's formula, $c' = 36{,}000$.
Keelhoff	.12	2.64	for all values of p.

Mr. Szlapka, in his design, made $A' = 1.15$ sq. ins.

The following list shows the value of $\frac{q-p}{q'}$ in chord A 9 L adjusted by multiplying the original values by the factors $\frac{\pi}{2}$ and 2 where necessary, for use in the formula

$$A' = k\,A\,\frac{2}{l}\,\frac{2\,r^2}{d}\,\frac{q-p}{q'};$$

Author	Value of $\frac{q-p}{q'}$	
Prandtl	.25	$p = 24{,}000$.
"	.42	$p = 13{,}571$ from $p = 16{,}000 - 70\,\frac{l}{r}$.
Engesser	.50	for all values of p.
Prichard	.27	" " "
"Modern Framed Structures"	.24	" straight-line formula.
"	.1070	" Rankine formula.
"	.0535	" Rankine formula.
Keelhoff	.19	"

The practical formulas thus give values for the net section of a lattice bar in chord A 9 L ranging from 0.75 sq. in. to 7.04 sq. ins. The rule in "Modern Framed Structures" is capable of giving values ranging from 0.75 sq. in. up to 336 sq. ins. The range of values is even more definite [?] than the numerical values indicate, depending as it does on the varying opinions regarding the values to be assigned to the constants of the column formulas.

It is evident that the number of rivets necessary to develop the values of the larger sections given above would make the use of lattice bars impossible. Cover plates and horizontal diaphragms would be required.

The value of $\frac{l}{r}$ for the outer web in chord A 9 L is 44, and for the column as a whole 34.7. This is not good design, as the first value ought to be less than the second.

The latticing between the center webs is inefficient. Intermediate latticing should have been used on these webs. One of the bars between the center webs in every panel on the upper face of the chord A 9 L has a net section of only 1.15 sq. ins. at the center and 1.5 sq. ins. for a length of 4½ ins., whereas between the center web and the outside web the section is 2.48 sq. ins.

The bending moments and shears in the webs, the bending moments in the latticing and the compressive stresses in the latticing due to the load on the column, have not been considered in this discussion. The theory of the design of latticing has been discussed on the assumption that the curvature in the column under load is negligible, as it ought to be.

When appreciable bending occurs, the total transverse shear is still given by the formula $S = P\,\theta$. On the other hand, when the curvature of the axis of the column varies from point to point, the longitudinal shear S' will not be as great in comparison with S as if the column remained straight, on account of part of the transverse shear being balanced by the resistance to bending of the webs taken individually. Only the difference between these actions is thrown into the latticing and represented by S'.

A method of dealing with the shears which in some respects is simpler than that adopted might have been used. This simpler method is based on the assumption that the small bending stresses at the ends of the webs in a panel of latticing, due to the assumed unity of the column, may be neglected. In this case q will denote the average unit stress in the outer web under eccentric loading and not the extreme unit stress in this web. This method has indeed been partially applied in this appendix. The results do not differ appreciably from those of the general method adopted.

FAILURE OF LOWER CHORD A 9 L.

In discussing this failure the original conditions will be assumed to hold, that is $S' = 0.54\,S$ and $0.45\,S$ between the inner webs and between the inner and outer webs, respectively. It is possible that these two values were closer together owing to the working loose of the latticing between the inner webs.

Assume P = the load at the time of failure = 14,000,000 lbs.; and P' = 50,000 lbs., a load sufficient, according to experiments made at Philadelphia, to cause slow movement and rivet slip,

$$\text{then } S = \frac{50{,}000}{.54} = 92{,}592 \text{ lbs. per sq. in.}$$

Now $S = P\,\theta$

$$\text{Therefore } \theta = \frac{S}{P} = \frac{92{,}592}{14{,}000{,}000} = .0066.$$

Thus if the obliquity = .0066 existed under a load P = 14,000,000 lbs., the chord would gradually go to destruction.

The measurements made by Messrs. Birkmire and Kinloch on Aug. 27, 1907, show, when averaged up for the four webs, a deflection of the chord as a whole of 1¾ ins. at the point between the second and third lattice panels from the south end. Since no measurements were made to determine the position of the ends of the chord from panel point to panel point it is impossible to state the real deflection of the chord, and the only assumption which can be made is that it is represented by the above amount.

It is not possible to state why the maximum deflection took place at the point mentioned. There may have been an original deflection of small amount there, a defect in workmanship or a local injury from the fall mentioned in Appendix 11. The accompanying buckling of chords 8 R and 9 R of the cantilever-arm [see Figs. 5 and 6, p. 458, Eng. News, April 23, 1908] shows that the failure itself was not accidental, although it may have been localized by defects or accident. The Philadelphia tests show that slip in the lattice system would have commenced with an obliquity less than one-half of that obtained above, and it is possible that the first movement was caused by a combined stress due to a small initial obliquity and to the shortening of the chord under stress.

It is hardly necessary to note that, owing to the form taken by the chords, partial failure in the lattice system must have preceded any failure in the web system.

If now it be assumed that the chord had an original deflection of ½-in. at the place in question, the inclination of the axis of the chord in the first panel of latticing will be found to be $\frac{0.5}{191} = 0.0026$, which is greater than in any other panel. To make up the required obliquity of .0066 it will therefore be necessary to assume that the direction of the load originally had an obliquity of $.0066 - .0026 = .0040$ to the axis. This would be equivalent to 2¾ ins. in the length of the chord, probably due to an eccentricity of about ½-in. to the east at panel point 8-9 and 2¼ ins. to the west at panel point 9-10.

From the discussion given in this Appendix and from the results obtained in the test of model chord No. 1 [Eng. News, April 23, 1908] it is not difficult to see that failure was certain and close at hand on Aug. 27. The evidence shows that the increase of obliquity which created this danger condition took place between Aug. 24 and Aug. 27.

On Aug. 27 the curvature was such that the line of load which would give the least maximum obliquities in the chord had an eccentricity with regard to the center line adopted for the measurements towards the west of 1¾ ins. at panel point 8-9 and of ¾-in. to the east at panel point 9-10. As this line of load gives minimum lattice stresses we assume it, for the purposes of this investigation, to be the true line of load. The inclination of the axis of the chord in the first panel of latticing at the south end with reference to the same center line was about .016, thus making the obliquity of the line of load in this panel about $.016 - .004 = .012$.

The question now occurs how was it possible that the chord could sustain an obliquity of .012 when an obliquity of .0066 was sufficient to strain the lattices to the danger point?

If the chord had remained straight an obliquity of $.0066 \times \frac{60{,}000}{50{,}000} = .0079$ would have caused immediate failure.

In reply it may be said, as has already been pointed out, that the consideration of the bending moments in the individual webs accompanying the bending of the chord, the bending moments in the latticing, the compressive stresses in the latticing due to the load on the columns, etc., has heretofore been omitted. Of these the first appears to be the most important and its effect in aiding the lattice system may be estimated as follows:

Let M denote the increase in the bending moments of the outer web in the first panel of latticing at the south end of the chord, M' the corresponding quantity for the inner web

then

$$54.36\,S' = 69\,S - 2\,(M + M')$$

$$\therefore S' = \frac{69\,S - 2\,(M + M')}{54.36}$$

the length of this panel being 69 ins.

Now the chord in the length of the first two panels of latticing, viz., in 142 ins., has a central deflection of ¼-in. The radius at the middle point may thus be computed approximately. The resulting value is $r = 10{,}000$ ins. The true radius may be less than this as the web was probably nearly straight next the cover plate with increasing curvature towards the point of greatest deflection. It is even possible that there was a point of contraflexure near the edge of the cover plate.

Now
$$M = \frac{E\,I}{r} = \frac{30{,}000{,}000 \times 366}{10{,}000} = 1{,}098{,}000 \text{ in.-lbs.}$$
$$M' = \frac{E\,I'}{r'} = \frac{30{,}000{,}000 \times 239}{10{,}000} = 717{,}000 \text{ in.-lbs.}$$

From what has been said with reference to the change of curvature along the length of the chord, it is not unreasonable to assume that the above bending moments represent approximately the increase of bending moment in the length of the first panel of latticing, i. e., in 69 ins. from south to north.

Thus
$$2\,(M + M') = 2\,(1{,}098{,}000 + 717.000) = 3{,}630{,}000$$
$$\therefore\ S' = \frac{69\,S - 3{,}630{,}000}{54.36}.$$

Now
$$S = P\,\theta = 14{,}000{,}000 \times .012 = 168{,}000$$
$$\therefore\ S' = \frac{69 \times 168{,}000 - 3{,}630{,}000}{54.36} = 146{,}468.$$

And
$$P' = \frac{1.4}{4}\,S' = .35 \times 146{,}468 = 51{,}264 \text{ lbs.}$$

Thus on account of the resistance to bending of the individual webs, the obliquity .012 will produce a stress in the lattice bars of about 51,264 lbs., whereas had the webs remained straight, an obliquity of only .0079 would have destroyed the lattice bars. In the tension experiments described in Appendix 15 on lattice bars like those used in the Quebec Bridge the breaking values, or P', were 60,100, 59,800 and 59,500 lbs.

In the above calculations the compressive stresses in the lattice bars due to the compression of the chord as a whole have been neglected.

The above explanation of the failure of this chord under three-quarters of its maximum working load contains assumptions which render it only tentative. It indicates the dangerous effects of even small obliquities and deflections on the safety of a chord with weak latticing. It is quite probable that the obliquity was in great measure due to movements at the field joint in panel 9 L, which was riveted up, and at the field joint in panel 10 L which was being riveted up at the time of the collapse. In fact all the troubles in the lower chords of both anchor and cantilever arms which developed after Aug. 6, 1907, seem to be partly attributable to movement at the field joints. These movements were noticed principally in the inner webs, which have much less horizontal stiffness than the outer webs. These webs were intended to carry the same unit-loads as the outer webs, and yet at the field joints they were connected to the cover plates with only half as many rivets, the small web angles used not permitting more. The outer webs with heavy angles and fairly effective latticing seem to have stood up under the stresses; the small angles and inefficient splicing and latticing of the inner webs allowed them to yield, thus disturbing the intended action at the field joints and panel points and giving opportunities for unforeseen eccentricities of loading. Heavier angles on the center webs under the cover plates, heavier splicing and heavier top and bottom cover plates would have added much to the efficiency of the joints.

An important function of cover plates is that they maintain the webs or ribs at their proper distances apart, but in erection, the bottom cover plate was taken off during the riveting up of the joint, and was replaced by small angle bars which were entirely too slight to perform the function of the cover plate. This is shown by the fact that a much greater movement was noticed at the bottom of the center webs than at the top.

TEST OF MODEL CHORD NO. 1.—This chord broke without warning under a load $P = 2{,}322{,}600$ lbs., by the failure of the outside lattice rivets. The ultimate shearing value of one rivet was 4,000 lbs. The lattice angle was connected with the web by two rivets. From the foregoing formulas, therefore,
$$S = \frac{P'}{k} = \frac{8{,}000}{.45} = 17{,}778 \text{ lbs.}$$
$$\theta = \frac{S}{P} = \frac{17{,}778}{2{,}322{,}600} = .0077.$$

The obliquity of the load which caused the failure was thus .0077, subject to correction for error of calibration of the testing machine.

TEST OF MODEL CHORD NO. 2.—The test of chord No. 1 showed that the lattice system was too light but gave no indication of the ultimate strength of the column if properly latticed. The capacity of the Phoenix Iron Company's machine was not sufficient to permit a complete test of this kind. In order, therefore, to get results, a chord with only two webs was constructed. The lattice system, however, was made about twice the strength of that in model chord No. 1 and the length of the model was only 11 ft. 4¼ ins. c. to c. of pinholes. The lattice bars were connected to the web by four rivets instead of two. This chord fulfilled the expectations of the Commission and broke under a load of 37,000 lbs. per sq. in. by the yielding of the webs in the center panel.

From what has been said it is evident that this experiment did not settle the question of the strength of the latticing. Stronger latticing might have been required in good design. The proper inference is that the obliquity was too small to break the latticing, so that the full strength of the webs was nearly, if not quite, developed.

Since the inside webs of the Quebec chords are less stiff than the outer webs, it seems to be a fair inference that 37,000 lbs. per sq. in. is higher than the strength of the Quebec chord would have been, even if properly latticed. Some allowance also must be made for the higher strength and elastic limit of the small plates and angles used in these models as compared with those in the bridge. There is doubt as to the correctness of the calibration of the testing machine, so that the above figures are subject to correction. In the tests of both models, the dishing of the webs between the upper and lower lattice systems was small and only careful measurements rendered its existence apparent.

In concluding this appendix, some brief comment is necessary upon two points:

1. *The use made by Mr. Szlapka of the information existing in* 1903 *respecting the design of latticing*: It has been admitted by Mr. Cooper that he failed to give the design of the lower chords the degree of personal attention that he gave to the details of the tension system. The foregoing discussion shows that even at the present time theories of lattice design are seriously in conflict and the strength of any lattice system will vary materially according to the formula adopted. Mr. Szlapka used, with his own modifications, the only system of lattice computation generally known to American engineers. This method involved the choice of a column formula from which to determine certain quantities necessary in the lattice computations. Mr. Szlapka selected the column formula adopted by his own company, and used the constants for it that, in his judgment, were most in keeping with the conditions of the case and in best accord with the spirit of the specification. He made what he considered a liberal increase in his adopted sections over what his computations called for. The result has shown that his judgment was faulty, but we are not prepared at this date to define the maximum safe sections for the latticing for these chords. The profession has learned much from Mr. Szlapka's mistake, but it is not even yet in a position to determine the percentage of his error. The lattices of Model Chord No. 2 were proportionately only 50% heavier than those used on the Quebec chords and yet they did not fail until the webs buckled. We have indicated in the discussion that Mr. Szlapka's attention would soon have been drawn to the weakness of the theory by which he was guided, had he made any study of the results given by that theory with different assumptions. No explanation, except the previous uniform success of compression members in service, can be offered for his failure to do this.

2. *The application in practice of the theoretical formula given in our discussion*: This depends upon our ability to select values of q and p suitable to the detail of construction in the special column under consideration. The values of p are determined in practice by the use of column formulas, but no one contends that the range of the tests upon which these formulas are based is sufficiently extensive to cover all the conditions that affect column strength; the formulas are simply accepted as the best guide that we now have. The maximum value of q will be fixed by the characteristics of the metal to be used. It is evident that by experience values of q and p may be gradually determined which will make it possible to design latticing that will be unquestionably safe and not unnecessarily heavy. We may here point out that great compression members, such as the Quebec Bridge chords, call for just as much individual study in design as an ordinary small bridge, and that any specification for such members should give reasonable latitude for the exercise of judgment by the designing engineer.

Typical Compression Chords of Large Cantilever Bridges.

[Appendix 17.]

A useful comparison of large cantilever bridges is presented by the Commission in two tables, which we have headed Table I. and Table II. The former, in Appendix 17, gives data of typical compression chords of six large cantilever bridges, the Memphis, Thebes, Mononganela, Blackwell's Island, Forth and Quebec bridges; Table II., forming part of Appendix 18, is concerned with these bridges themselves. The two tables are supplemented by outline drawings of the structures and detail drawings of the compression chords selected for comparison. These drawings we have grouped on an inset sheet in this issue.

The outline drawings of the bridges, and the information of Table II. as to loading, etc., should first be referred to. As the Commission remarks, only the Forth Bridge is comparable with the Quebec in point of span [1,710 ft. and 1,800 ft. respectively], but the heavy loading for which the Blackwell's Island Bridge is designed makes it the heaviest of all, so that its compression chords are to be grouped with those of the two longer-span structures. It happens also that the Blackwell's Island Bridge, the only uncompleted one of the six except the Quebec Bridge, employs very high unit-stresses, and its compression chords are rather similar in design to the Quebec chords. So far as we are aware, no tests have been made to verify the assumed strength of the Blackwell's Island chords or to show that they are free from the weaknesses revealed by the Quebec collapse.

The chords detailed in Table I. and shown by the drawings on our inset sheet are not the largest compression members of the respective bridges. Their position in the bridge in each case is marked on the outline drawings by drawing the chord in question heavier than the others. The Forth Bridge is an exception to this; here the detail drawing is a typical sketch applicable to all the compression members, and does not refer to any one chord-section. For the other bridges, however, the chords shown are equally typical, only the make-up [and in some cases the precise length] being special to the chord marked on the outline plan.

In the table we introduce for use in comparison, an example giving the dimensions of an ordinary bridge post of the two-channel type, the figures being taken from Professor Burr's "Elasticity and Resistance of the Materials of Engineering." These dimensions are more or less typical of those latticed columns that have been used in bridge construction with such success during the last twenty-five years; the details of such columns are now designed entirely by practical rules.

It will be noted that the Forth Bridge chord is in a class by itself. It is not a latticed section, but may be regarded as a solid section built up out of separate plates. No criticism touching the practical success of this design has ever been made, but it is not a class of construction that could be adopted by an American bridge company without making material changes in its shop equipment and methods of handling its business. We have, however, noted in Appendix No. 18 that the work of the Forth Bridge designers is worthy of careful study.

The examples taken from American practice may be divided into three groups:

(1) Chords of the ordinary two-channel type which reaches its maximum development in the Monongahela design.

(2) Chords of the four-channel type, latticed into one column, as adopted for the Memphis and Quebec bridges.

(3) Chords of the four-channel type, latticed into two columns which are made to act together by means of tie-plate connections. This type was adopted for the Thebes and Blackwell's Island bridges.

In the table herewith we give the principal dimensions of the chords shown on the drawings.

It is almost impossible to find any common basis for a comparison of these chords. It must be remembered that latticing is often uniform in size in members on the same bridge doing similar service, but having different loads and cross sections. Thus in the Quebec Bridge, chord A 9 L had an area of 781 sq. ins.; and chord A 1 L an area of 301 sq. ins., yet both members had about the same outside dimensions in cross-section, and the same latticing. Therefore, as the chords selected for the drawings are not the most heavily stressed chords in the respective bridges, comparison by proportion of lattice to main sections would be unfair. In fact, we may say that the drawings given are only typical.

A theoretical comparison between the lattice systems of the different columns might be made by using any one of the various formulas given in Appendix 16, but we have already pointed out that no one of these formulas is generally accepted by the profession. There are so many causes of variation in the strength of built-up chords of equal area which are not provided for in these formulas, that comparison by calculation does not appear to be satisfactory.

Referring to the table it will be noted that the Quebec chord has considerably less horizontal stiffness (see values of $1/r$) less lattice area, less rivet area, and less splice-plate area in proportion to the size of the members than any of the earlier bridges. It should be remembered also that the unit stresses for the Quebec Bridge were higher than those of the earlier bridges.

It will be noted that the earlier designers considerably overran the 15 or 20% of splice-bar area. This is also true of the Quebec Bridge chords, but not to the same extent. Mr. Szlapka states that splice plates having an area of cross-section equal to 15% or 20% of the cross-section of the member would be satisfactory.

The development of the detail plans of the Blackwell's Island Bridge was contemporaneous with that of the Quebec Bridge plans; the Quebec designers had not access to the Blackwell's Island plans. In fairness to the Que-

TABLE I.—COMPRESSION CHORDS OF LARGE CANTILEVER BRIDGES.
[See also Table of Dimensions of Cantilever Bridges, in Appendix 18.]

Bridge.	Area of cross section sq. ins.	Sectional area of lattice* sq. ins.	Sectional area of latt. rivets† sq. ins.	Length of chord, l ft. ins.	Rad. of Gyr.‡ r ins.	Length ratio l/r.	Weights per lin. ft. of chord. Main section, lbs.	Weights per lin. ft. of chord. Lattice, lbs.	Dimensions of chord. Depth b. to b. angles, ft. ins.	Dimensions of chord. Width o. to o. web-pls, ft. ins.	Sectional area of Horiz. splice pls. sq. ins.	Sectional area of Vert. splice pls. sq. ins.
Quebec	781	10	4.8	57 0	19.7	35	2,603	66	4 6½	4 9⅜	68	212
Memphis	213	10	4.8	28 2¾	14.8	23	710	48	2 6¾	3 2	46	250
Blackwell's Island	852	25	4.8	31 6½	22	17	2,840	144	4 0½	5 2⅝	80	180
Thebes	189	11¼	4.8	30 6⅝	20.1	18	630	78	3 0½	4 4¾	80	70
Monongahela	262	14⅛	7.2	30 6⅝	25.4	14	873	82	3 0¾	4 1	74	161
"Burr"	35	3¾	1.2	45 0	6.5	83	118	20	1 6	1 0¼	—	

*Total area of lattice members cut by a cross-section of chord; area measured at right angles to axis of lattice-bar.
†Area of rivets connecting all lattices cut by a cross-section of chord to outside web, one end only.
‡Radius of gyration in a plane parallel to latticing (axis normal to latticing); not necessarily the least radius.
Note: The horizontal stiffness of the inside ribs of the Quebec Bridge chords is less than that of the outside ribs, which is not the case in any of the chords of the other bridges.

bec Bridge designers, however, it should be pointed out that in the Blackwell's Island Bridge the proportions of many of the details are much more nearly in accord with Quebec Bridge practice than are those of the earlier bridges, although the principles of the designs are very different.

A consideration of the difference in the designs [shown by the drawings on our inset sheet], all of which have been prepared under the direction of engineers of recognized ability and high professional standing, shows that there is as yet no established system of design for large compression members. The individual judgment of the engineer is the determining factor, and this may prove to be erroneous, as it did in the case of the Quebec Bridge.

The lack of precise knowledge on this subject has been discussed in other appendices.

A Critical Discussion of Certain Parts of the Specifications.

[Appendix 18.]

. . . Specifications as a rule consist of two distinct portions, one of which relates to design and the other to fabrication, material and execution. In the case of the Quebec Bridge, the difficulty of preparing an adequate specification for design was very great. It would have been better to have entrusted the preparation of the plans and specifications to engineers independent of any contracting or manufacturing company, whose previous experience qualified them to handle the work. This course would have avoided duplication of designs involving expensive plans, and would have prevented the letting of a contract on incomplete plans formed upon erroneous data; the engineers would have made a proper and sufficient study of the whole project, and in due time competitive tenders upon their plans would have been secured, thus enabling all contractors to tender on a common basis. The privilege of submitting independent plans might have been extended to the bidders.

. . . An error of judgment made by the Quebec Bridge Company was the selecting of an engineer who did not possess the necessary special knowledge and experience to prepare the specification (See Appendix No. 7). It is true that this specification was considered to be only tentative, drawn up for the purpose of procuring preliminary tenders, but its history and importance cannot be overlooked. (See Appendix No. 6.) It became the basis of the contracts between the Quebec Bridge Co. and its contractors, was approved by the Government engineers, and was an essential part of the Subsidy Agreement whereby the Dominion Government undertook to pay the Quebec Bridge Company on certain conditions $1,000,000.

The specification itself, herein called the 1898 specification, was for the most part a copy of a specification issued by the Department of Railways and Canals in 1896; there is nothing in its wording to indicate that the Quebec Bridge was an exceptional structure and without precedent, or that the propriety of applying to this structure other than the usual clauses in bridge specifications was carefully considered.

In regular bridge practice the specification is of importance particularly because an American bridge works is a factory for turning out structural steel fabricated in accordance with plans prepared in the drawing office attached to the works. This drawing office is a part of the factory, and in it, as throughout, efficiency is obtained by standardizing and duplication; the drawing office staff consists of a number of well-trained computers and draftsmen whose duty it is to prepare the shop drawings for the works and who are under the control and direction of a designing engineer. Details are designed in accordance with the specifications furnished by the purchaser, except under circumstances when shop equipment requires some deviation to be made to secure facility of manufacture. It is not a part of the duty of the drawing office staff to question the wisdom of the requirements of the specification, nor could the progress of work throughout the factory be satisfactorily maintained if it should attempt to do so.

The evidence shows that the Phoenix Bridge Co. followed this usual practice in the preparation of the Quebec Bridge designs.

In 1903 it became necessary to design the main spans of the bridge and the 1898 specification was amended by Mr. Cooper, it having been understood ever since 1900 that it would be amended and altered. Mr. Cooper did not recognize these amendments as complete and final, and considered that he had the power to deal with each problem of design as it arose, and he exercised this power when he thought necessary. The designing of the main spans was left to Mr. Szlapka, Mr. Cooper having approved the specifications, and no one questioned any decision that these engineers made. The work was done under the immediate direction of Mr. Szlapka.

Before discussing the specification it will be well to contrast some of the main features of the Quebec Bridge with those of other great cantilever bridges. One table herewith [Table II.] is inserted for this purpose.

It is not possible to set forth all the facts in a table with sufficient minuteness to justify the making of complete comparisons, as many qualifying clauses and special conditions are necessarily omitted. Three items of interest may be noted in the table:

1. Only the Forth Bridge is at all comparable with the Quebec bridge in regard to span.

2. Only the Blackwell's Island Bridge is comparable with the Quebec Bridge in regard to unit stresses, both for main members and for details.

3. All the bridges included in the table were designed by independent engineers except the Quebec Bridge.

In this connection we must express the opinion that it is difficult for the employees of a large manufacturing concern to give the design of a bridge of unique features the concentrated attention that it requires.

With regard to precedent only the Forth and Blackwell's Island bridges involved anything like the same total stresses as the Quebec Bridge. The design and construction of the Blackwell's Island bridge was contemporaneous with the Quebec Bridge.

The Forth Bridge was built on a system not suited to the established American methods of bridge construction, so that its distinctive features of design, construction and erection were not followed. It is proper to add that the achievements of the Forth Bridge engineers deserve much closer study than appears to have been given to them on this continent. Messrs. Baker and Fowler succeeded in erecting a structure which weighs considerably less per lineal foot than the Quebec Bridge and which is designed to carry about one-half the rolling load and several times the wind load specified for the Quebec Bridge. The main compression chords of the two bridges are of practically equal area, but the material in the Forth Bridge is of a considerably higher ultimate strength than that used in the Quebec Bridge, the unit stresses are less, and the design of the cross-section of the chords is such that they should be able to carry a greater unit stress with safety. On great bridges these are factors worth observing, and it is to be regretted that the stress sheets and full engineering studies in connection with the Forth Bridge have not been published.

It is evident that the designers of the Quebec Bridge were compelled to work from experience gained on much smaller bridges.

In discussing the specification we deal not only with

TABLE II.—INFORMATION CONCERNING GREAT CANTILEVER BRIDGES.

Name.	Date of construction.	Designer.	Contractor for superstructure.	Span, ft.	Width c. to c. trusses, feet.	Live-load, lbs. per lin. ft.	Ult. tensile strength spec. for steel, in 1,000 = lb. units.*	Approx. weight of steel.† Tons (2,000 lbs.) per lin. ft.	Price of steel, cents per lb.‡	Allowed working stresses, lbs. per sq. in.	Allowed shear on rivets, lbs. per sq. in.
Forth	1882-1889	Baker & Fowler	Wm. Arrol & Co.	1710	Varying, lower chord 31½ ft. at ends, 120 ft. at piers.	Double track Ry.; 2240 lbs. per track.	Compr. 76-83 Tens. 67-74	10½	6.50	Max. stresses: Compression 17,000 Tension 16,350	About 12,000
Memphis	1886-1892	Geo. S. Morison	Union Bridge Co.	790	30	Single track Ry.; 4,000 lbs. per track.	Compr. 69-78½ Tens. 66-75	3½	5.88	Compr. 14,000 if $l < 16d$. Deduct 750 lbs. for each additional unit over 16 in $1 \div d$; tension for dead load 20,000, tension live load 10,000.	7,500
Monongahela	1902-1903	Boller & Hodge	American Bridge Co.	812	32	Double track Ry.; 4,500 lbs. per track.	Compr. 60-70 Tens. 63-75	4½	4.3	Compress. dead load 21,000 where $l/r < 40$. Tension dead load 22,000. Take one-half in each case for live load.	10,000
Thebes	1902-1905	Noble & Modjeski	American Bridge Co.	671	32	Double track Ry.; 5,000 lbs. per track, less 20%.	62-72	5	5¼ (not official)	Compress. dead load 21,000 if $l/d < 16$. Tension 20,000. Take ⅛ in each case for live load.	7,500
Blackwell's Island	1901-1908	Dept. Bridges New York City	Pennsylvania Steel Co.	1182	60	Roadway & trolley; ordinary 8,000 lbs. congested 16,000 lbs.	Compr. Approx.: 60-64 Tens. 66-70 Nickel-steel eyebars 85.	13½	5‡ (not official)	Compr.: ordinary 20,000 - 90 l/r. Congested 24,000 — 100 l/r. Tens. ordinary 20,000, congested 24,000. Tens. for nickel-steel: ordinary 30,000, congested 39,000.	Ordinary 13,000 Congested 16,000
Quebec	1900	Phoenix Bridge Co.	Phoenix Bridge Co.	1800	67	Double track Ry.; Roadway & trolley; 4,000 lbs. per track; for extreme conditions mult. 1⅛.	Compr. 60-70 Tens. 62-70	13	5.60	Compress: ordinary, 12,000 $\left(1 + \frac{\text{Min.}}{\text{Max.}}\right)$, extreme 24,000; both for $l/r < 50$. Tension: ordinary 12,000 $\left(1 + \frac{\text{Min.}}{\text{Max.}}\right)$, extreme 24,000.	¼ working stress = 18,000 extreme.

*The requirements given are for tests of small-sized specimens. Where figures for full-size tests were specified, they seem to have been changed to corresponding specimen-test figures for this table.—Ed.
†These appear to include weight of trusses, bracing, floorbeams and stringers, only.—Ed.
‡These prices are probably for steel erected.—Ed.

the clauses immediately connected with the downfall, but with others that were not in our judgment calculated to secure a safe and satisfactory structure.

The specification is here understood to mean the 1898 specification as amended in writing by Mr. Cooper.

As a document the specification is unsatisfactory, some of the clauses having been amended by Mr. Cooper, some set aside in favor of his well-known and generally accepted standard specification, and some remaining in force with a context that altered their meaning. No general or complete revision of the specification embodying Mr. Cooper's amendments was ever compiled.

As a matter of fact, although the 1898 specification was received as the official specification and much of the work done in accordance with it, we believe that Mr. Cooper depended upon his own inspection of the plans under the revised specifications to secure satisfactory details. His opinions upon most debatable questions of design were well known to the staff of the Phoenix Bridge Co., which had previously designed and built many structures under his direction and was accustomed to his methods. It is on record that the Phoenix Bridge Co. requested Mr. Cooper to set aside the 1898 specifications altogether and to substitute for them his own standard specifications.

A complete bridge specification must set forth the character of the material that is to be used, the loadings that are to be carried, the stresses to be permitted in the members and provisions concerning details, fabrication and erection to be observed; in fact, everything essential to the proper carrying out of the work as intended.

MATERIAL.—The 1898 specification for material was used without alteration except in one particular. Mr. Cooper having raised the minimum limit for the ultimate strength of eyebar material from 60,000 to 62,000 lbs. per sq. in. The metal specified was the ordinary grade of structural steel.

It will be noted by reference to the table that the Quebec Bridge specification called for material of slightly lower ultimate strength than that used in any of the other bridges, while the bridge itself had the longest span of all. The need of a better material than structural steel for the construction of long-span bridges is generally recognized, because the decrease of total weight and consequently of cost in a large truss with increase of permissible unit stress is very rapid. In the Quebec Bridge the dead-load stresses constituted roughly two-thirds of the stress on the main members.

The designers of the other two great bridges introduced special grades of steel, so that high unit stresses could be safely used. The Forth Bridge engineers were not permitted to load their metal to more than one-fourth of its tensile strength, and for compression used a steel about 25% stronger than that supplied for Quebec. Nickel-steel with a permissible unit stress 50% higher than allowed on material in the same bridge and similar to that used at Quebec was introduced into bridge practice by the Blackwell's Island Bridge designers. The use of this alloy as a structural material was investigated and favorably reported upon in 1903 by a special commission of which Mr. Cooper was a member.

It was Mr. Cooper's opinion that it was wiser to use the ordinary grade of metal for the Quebec Bridge and to load it to the highest working stresses that were considered practically safe.

ELASTIC LIMIT: We do not know whether Mr. Cooper in his amendments intended the term "elastic limit" to mean the elastic limit of a test specimen or of a full-sized member. There is also some uncertainty as to the true meaning of the term "elastic limit," which is unfortunate, as the maximum working stresses specified are made to depend upon this characteristic of the material.

The "elastic limit" accepted by bridge designers as a controlling factor in their work cannot be determined by the method prescribed by the 1898 specification and yet this method (the drop of the beam) was used. Both Mr. Cooper in his standard specifications and the engineers for the Blackwell's Island Bridge provide for a much closer determination of this characteristic. In reality the determination is a delicate and time-consuming process for a research laboratory and impossible under the conditions existing in a rolling mill; to such an extent is this true that it is not called for in the carefully prepared specification issued by the American Railway Engineering and Maintenance of Way Association in 1906. The principle apparently followed in the latter specification is that mill tests are sufficient for mill purposes and that the true elastic limit can be most safely obtained by proportion from the ultimate strength. The assumption generally made is that the true elastic limit for structural steel is about 50% of the ultimate strength.

The material actually supplied for the bridge was regularly tested and a comparison between its probable elastic limit and the 32,000 lbs. per sq. in. apparently expected by Mr. Cooper, is possible. The full-size eyebar tests show that the metal in service shape had a safe ultimate strength not in excess of 55,000 lbs. per sq. in. and a reported elastic limit of 28,000 lbs. per sq. in. These tests were made on long bars in the Phoenix Iron Co.'s large testing machine, and the results might be reduced by calibration of the machine and closer observation of the elastic limit. It will be noted that the proposed extreme working stresses (24,000 lbs. per sq. in.) for the Quebec Bridge were nearly equal to the elastic limit of the eyebars.

The elastic limit in compression was assumed in accordance with the usual practice to be the same as that in tension. An examination of the voluminous test records shows that an ultimate strength in excess of 60,000 lbs. per sq. in. was not regularly secured, so that, accepting the 50% relation mentioned above, the elastic limit in compression becomes 30,000 lbs. per sq. in.

It should be noted that these tests were made on specimens of about ½ sq. in. sectional area. The compression members were built up of wide thin plates riveted together into webs. We know of no test that has ever been made to establish the relation between the strength and elastic limit of such plates and those of small test specimens, nor do we know what effect the punching, riveting and painting have on the material in the webs as compared with the solid plate. It was noted at the wreck that the paint between the plates of members that had been fabricated for over three years was still fluid. From the analysis of full-sized tension tests we think it possible that the elastic limit of the plates in the compression members was not much above 27,000 lbs. per sq. in. instead of 32,000 lbs. as apparently assumed.

UNIT-STRESSES: The maximum unit-stresses that Mr. Cooper proposed to use were about 21,000 lbs. per sq. in. under ordinary loading and 24,000 lbs. per sq. in. under extreme conditions. He considered that the extreme conditions as specified would never occur.

By reference to the table it will be seen that the *specified stresses for the Quebec Bridge under working conditions are in advance of current practice and we believe that they are without precedent in the history of bridge engineering.* Under extreme conditions the Quebec Bridge stresses are in general harmony with those permitted in the Blackwell's Island Bridge.

We have already indicated that the dimensions of the Quebec Bridge were such that the use of the highest safe unit-stresses was justifiable and good engineering practice. If we were sure that the loads were correctly estimated, that the stresses acted in the bridge exactly in accordance with the assumptions and that the elastic limit of the built-up members was not less than 32,000 lbs. per sq. in., 24,000 lbs. per sq. in. would not be an unsafe stress for structural steel, provided that the material is regular in quality and the details satisfactorily worked out to suit such a stress.

Mr. Cooper provided for the effect of live-load by the use of the so-called fatigue* or $\frac{\min}{\max}$ formula. This method, which was formerly much used, has more recently been abandoned in general practice and is not adopted by Mr. Cooper in his standard specifications. In the hands of an experienced engineer this method will be made to produce much the same results as the more modern impact formulas. We do not know why this formula was used in this case, except that it was adopted by Mr. Hoare in 1898 from the 1896 specifications of the Department of Railways and Canals and was probably retained in 1903 for convenience.

Mr. Cooper adopted the ordinary straight-line formula for compression members, making the dead load unit stress equal to $\left(24{,}000 - 100\,\frac{l}{r}\right)$ lbs. per sq. in. We have already indicated in Appendix No. 13 that this formula is purely empirical and does not agree particularly well with the recorded tests upon large columns. It is the most generally accepted formula of practice, but we do not believe that the engineering profession has at present a satisfactory knowledge of the action of large steel columns.

There is a wide field for experiment which must be worked over before engineers can claim to have a sufficient knowledge of steel to design both safely and economically, and perhaps the most serious criticism of the structural engineers of the present day is that they have permitted this field to remain undeveloped for twenty-five years, during which time they have adopted a new metal for their work and new shapes and sections.

We think that in popular engineering opinion the ultimate strength of steel columns is largely overestimated, the diagrams [see Eng. News, April 9, pp. 404-5, Figs. 1-3] indicating that for the Quebec chords it was not safe to expect an ultimate strength in excess of 32,000 lbs. per sq. in., so that under the extreme conditions specified the margin of safety would only have been one-third.

*The allowance is by no means for fatigue, but rather represents a recognition of the increased effect of live-load as compared with dead-load. It is not a true impact allowance, as it does not depend on loaded length. Mr. Cooper does not admit the existence of fatigue as regards working stresses.—Ed.

This is a point on which current engineering practice is open to direct criticism. The older engineers, upon the results of whose experiments the profession is now depending, did not think of loading metal in compression to the unit-stress used in tension because they recognized that the ultimate unit-strength of members in compression was far less than that of members in tension.

The later school of engineers seems to have adopted the principle that the action of bridge members under stresses in excess of the elastic limit is a matter of indifference, as they will never be so stressed. The action within the elastic limit being practically the same under both conditions, they adopt the same working stresses in tension and in compression. Their practice has been attended with complete success, but this may be attributed to the fact that the material has ordinarily not been stressed to much above half the elastic limit.

Under the Quebec Bridge conditions where high working stresses were imperative the wisdom of the practice of loading in compression as heavily as in tension becomes questionable. We believe that in no great public structure should stresses be permitted in excess of one-half of the ultimate strength of any compression member, no matter how high the elastic limit may be.

It will be noted that Mr. Cooper in specifying the stresses for the lower chords of the Quebec Bridge omitted the term in the column formula containing the ratio $\frac{l}{r}$. In this practice he is supported by the engineers of the Monongahela and Thebes bridges, who made a similar provision, but they reduced the maximum stress to that allowed by the usual formula for a column with $\frac{l}{r}$ equal about 40.

The failure of the Quebec chords does not prove that Mr. Cooper was theoretically incorrect and cannot be directly connected with this clause in the specification. The specification, however, permitted stresses in advance of any previous practice and the proportioning of columns to safely carry such stresses is yet to be learned.

We have already pointed out [Eng. News, March 19, p. 308, and April 16, p. 422] the seriousness of the error made in the estimation of the dead load which resulted in computed stresses nearly 10% higher than had been expected. A comparison of these computed stresses with the elastic limit of the material as estimated from the test records will show how narrow a margin of safety was provided in the actual design.

We are not prepared in the present status of the art of bridge building to approve the unit-stresses stated in the amended specification.

RIVET STRESSES: It will be noted from the table that the rivet stresses used were much in excess of previous practice. These seem to have been adopted almost by an oversight. The 1898 specification contained a clause usual in low-stress specifications, permitting the rivets to be worked to three-quarters of the allowed stress in the member. This clause was not cancelled by the 1903 amendments and under extreme conditions permitted a stress in rivet shear of 18,000 lbs. per sq. in. The tests made in 1904 under the direction of the American Railway Engineering and Maintenance of Way Association have established the fact that a riveted connection begins to work under a stress in rivet shear between 12,000 and 15,000 lbs. per sq. in. and that deformation in even a simple connection is marked when a stress of 25,000 lbs. per sq. in. is reached. These results have been confirmed both in tension and compression by the tests made for the Commissioners. It is therefore clear that the Quebec specification permitted the use of stresses in details which were outside the limits of established practice and are now known to be unsafe. Knowledge of the action of the rivets in riveted connections is very incomplete.

BUILT-UP COLUMNS: In our findings we have stated that the bridge failed through weakness of the lower chords and particularly in the latticing of these chords. In Appendix No. 16 will be found a discussion of lattice design and of the data that Mr. Szlapka had to guide him in his work. The main outline of the latticing in the Quebec Bridge was sketched as early as 1898. There was practically nothing in the specification that was of any service to the designers in this connection and they violated none of its provisions in the design. There are some clauses dealing with latticing, but they were copied from small bridge practice and were wholly inadequate for the Quebec structure. The main criticism that can be made of the designers was that they had the means of checking their theories by use of the testing machine and that they did not do this nor did they thoroughly study the possibilities of lattice formulas.

LOADINGS: In 1903 Mr. Cooper revised the loadings, increasing the specified train loads and decreasing the wind pressures. While Mr. Cooper undoubtedly made an improvement on the 1898 specification in this respect, he does not seem to have taken full advantage of the

improved financial situation due to the decision of the Government to guarantee the Quebec Bridge Co.'s securities. Mr. Cooper apparently did not realize the great change in the traffic conditions that would probably follow the opening of the National Transcontinental Railway, nor the demands for transportation resulting from the rapid development of Canada. His specified train loading is not greater than that used regularly in Canadian practice and is lighter than that subsequently adopted for the National Transcontinental Railway, and sufficient provision was not made for probable increases of live load.

Considering together the high unit-stresses permitted and the loads specified, the specification was not for a bridge well suited to the purposes it would have been called upon to serve.

DEAD-LOAD: The specification requires that the dead load used for calculating the stresses shall not be less than the actual weight of the structure when completed. The evidence shows that the designers failed to comply with this requirement. [See Fig. 1, p. 421, Eng. News, April 16, 1908.] In view of the high unit-stresses specified this error was serious enough to have required the condemnation of the bridge even if it had not failed from errors in the design of the compression chords. . . . The consequences of this error were considered by Mr. Szlapka and Mr. Cooper before erection was resumed in 1906 and they state that it was their opinion that the error was not fatal to the safety of the bridge, and the work of erection was proceeded with.

ERECTION: No special provision is made in the specification for an oversight of the methods of erection by the Quebec Bridge & Railway Company's engineer, or for his approval of the general system of erection, or of the means adopted to solve the various problems arising in connection with it. There is no evidence to show that anyone outside the Phoenix Bridge Company attempted to deal with this practical problem. Mr. Cooper states that the erection plans and devices were not subject to his approval, although he was advised of them unofficially and general progress on erection was regularly reported to him.

It was apparently intended, as is the usual practice, to leave all such arrangements in the hands of the contractor, making him provide all necessary plant and holding him responsible for everything that might happen.

The erection staff of a large construction company is best qualified by experience to design erection plant. We are of opinion, however, that the erection difficulties to be met with on a structure like the Quebec Bridge are so serious and the necessary risks to be run during erection are so great that if the employment of a bridge engineer is necessary at all, it is especially necessary in this connection. In fact, the responsible engineer on such a project should direct the work in all its branches and the contractor is entitled to look to him as a trained specialist for instructions and assistance at all times and especially in emergencies.

The specification throughout shows that the whole subject was not considered with sufficient care, not only from a technical standpoint, but from the practical or business standpoint as well. Inconsistencies are of frequent occurrence; ambiguity and lack of precise definition pervade the whole.

Wind Pressure and Deflections.

[Appendix 19.]

In order to show complete proof that the wrecking of the Quebec Bridge was not due to wind-action, even in a contributory manner, the Commission's report presents a diagram of the wind-velocities during several weeks preceding the collapse, as recorded by the anemometer which the Phoenix Bridge Co. had placed on top of the center-post. The recording part of the instrument was in the field office, at the end of the south approach, and was not affected by the collapse of the bridge. Its record, extending up to the moment of the fall, shows that the wind velocity on the afternoon of Aug. 29 was only about 25 mi. per hr., corresponding at most to a pressure of about 2 lbs. per sq. ft. [or, by the formula $p = 0.004\ V^2$, 2.5 lbs. per sq. ft.]. This pressure was considered negligible in determining the stresses existing in the trusses at the time of the accident, and was not counted in computing the stresses given in Fig. 2, p. 422, in our last issue [April 16, 1908].

The temperatures at the time of the wreck and for some time preceding are also shown, and are also moderate.

The greatest wind velocities observed at the Quebec Observatory during a period of years were also examined. They show that a pressure of 25 lbs. per sq. ft. was sufficient as basis for designing a bridge at the site.

Some figures of lateral deflections of the bridge during erection are also given in the report, though they have, perhaps, no direct relation to the subject of the inquiry. They are as follows:

LATERAL DEFLECTIONS, TAKEN AT MIDDLE OF FIRST TRANSVERSE STRUT ABOVE DECK BETWEEN POSTS P1 OF CANTILEVER ARM (450 FT. FROM MAIN PIER).

Nov. 12, 1906. Front leg of large traveler at post P1 of cantilever arm (450 ft. from main pier). Panel 2 of this arm partly erected.
Wind: East, 55 mi. per hr.
Deflection: 2½ ins.

Nov. 16, 1906. Traveler in same position. Panel 2 nearly completed.
Wind: East, 65 mi. per hr.
Deflection: 3½ ins.

Feb. 3, 1907. Front leg of large traveler at T0 (506¼ ft. from main pier). Cantilever arm complete.
Wind: West, 45 mi. per hr.
Deflection: 2 ins.

AUTOMATIC PUMPING OF SEWAGE AT FLOOD PERIODS, DAYTON, OHIO.*

The city of Dayton, O., lies at the confluence of several streams, each of which is of insignificant proportions in summer. In late winter or early spring, however, they are liable to pour down such floods that the great Miami River takes on the proportions of a large stream. To prevent the city from inundation levees are maintained, but of course these do not prevent cutting off the discharge of sewers by the rising waters or eliminate the backing up of river water into streets and cellars. To provide for such cases gates have been constructed, at the sewer outfalls, to be closed at flood periods while the sewage is lifted by automatic, electric motor-driven centrifugal pumps.

Up to the present time three pumping stations have been constructed in various sections of Dayton. The first of these is equipped with three 2,500 gal., submerged, centrifugal pumps, geared to three-phase, 60-cycle, 20-HP., induction motors.

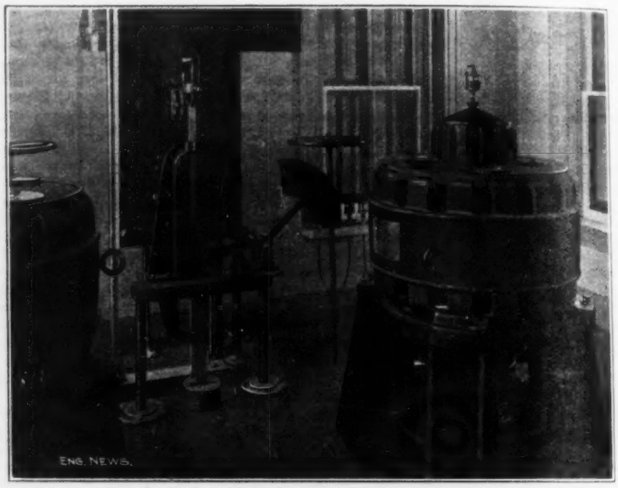
The motors are started and stopped by standard oil-switch compensators which are operated by hydraulic pistons controlled by float switches designed and built by the Dayton Hydraulic Machinery Co. under its patents. The lift is variable, averaging about 20 ft. The discharge is through a check valve at a point below extreme high water. The operation of these pumps is entirely automatic, it being only necessary to inspect them occasionally and keep the oil receptacles filled. Upon notice of a rise in the river, the gates are closed and the electric current turned on, after which the pumps cut in and cut out automatically with the rise or fall of the sewage in the pits.

A second station is equipped with two pumping units and the third plant has three. Each of these five units consists of a vertical double-suction, submerged, 4,500-gal., centrifugal pump, direct connected to a 40-HP., 60-cycle, three-phase, vertical induction motor.

The starting apparatus in these last two stations is worked primarily by float-operated valves on a cylinder which takes water, under pressure, from the city mains. A piston, in this hydraulic cylinder raises the lever of a Westinghouse auto-starter to the "starting" position, and at the same time rotates an arm, carrying a heavy counterweight, as shown in the accompanying figure. About the time the motors come up to speed this counterweight reaches a dead center position and falls over, throwing the lever on the starting panel to the "running" position. As the sewage is disposed of, so that the level falls to a predetermined point, the float-valves operate the piston so that a reverse motion is given the counterweight arm which in turn brings the starting lever on the panel to the "stop" position, cutting off current from the motors. The operation of this apparatus is reported as extremely satisfactory; the motors are cut in and out automatically with great regularity and smoothness. During one heavy rain, when the river was quite high, one pump was sufficient to take care of the flow in the sewers at one station by running three minutes out of each ten, being idle the remaining seven.

The pumps were built by the Dayton Hydraulic Machinery Co., of Dayton, O.; the induction motors and auto-starters were furnished by the Westinghouse Electric & Mfg. Co., of Pittsburg, Pa.

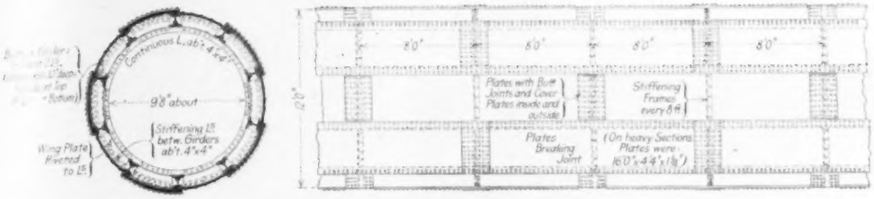
*From information furnished by the Westinghouse Electric & Manufacturing Co., Pittsburg, Pa.



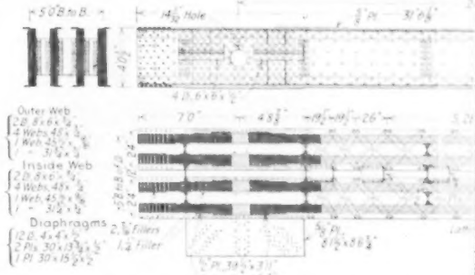
VERTICAL INDUCTION MOTOR AND STARTING MECHANISM; LEHMAN STREET SEWAGE PUMPING PLANT, DAYTON, O.

A CONCRETE ARCH OF 169 FT. SPAN, c. to c. of skewbacks has been built over the Sense River near Guggersbach, Switzerland. The arch carries a highway and was designed for a live-load of about 160 lbs. per sq. ft. and a 13-ton concentrated load. The roadway is a reinforced-concrete slab-and-beam construction resting on 24-in. reinforced cross walls spaced about 14 ft. apart. The arch has a rise of 28 ft. The ring is uniformly 13 ft. wide; its thickness is 44 ins. at the crown, increasing to 64 ins. at the skewbacks. The roadway plate is slightly cantilevered out on either side, the width between railings being 16½ ft. The arch was built in two layers, the lower one being concreted in separate blocks while the upper layer was placed in six sections separated by short keying blocks, immediately after the lower layer was completed. The arch was analyzed as an elastic structure with fixed ends; the maximum compressions are about 310 lbs. per sq. in. at the entrados at the crown, and 320 lbs. per sq. in. at the inner face at the skewbacks. The abutments are in rock, and are bedded at right angles to the arch axis. The maximum stresses in the reinforced portions are 500 lbs. per sq. in. in the concrete and 14,000 lbs. per sq. in. in the steel. The roadway plate and the back of the arch were covered with a layer of neat cement, to turn water; all other surfaces remained untouched. The roadway has expansion joints covered with metal sheets, over the skewbacks. The centering consisted of five ribs supported by nine transverse rows of piles. Sandboxes were used under the posts except those next the abutments, where wedges were provided. The bridge was built in 1906, by Gribl, Hassler & Co., of Burgdorf. The concreting of the arch ring required 32 days. The centers were not struck until three months afterward. The bridge cost 52,500 f. (about $10,500). It is described in the "Schweizerische Bauzeitung" of Feb. 29, 1908, by Mr. J. Bolliger.

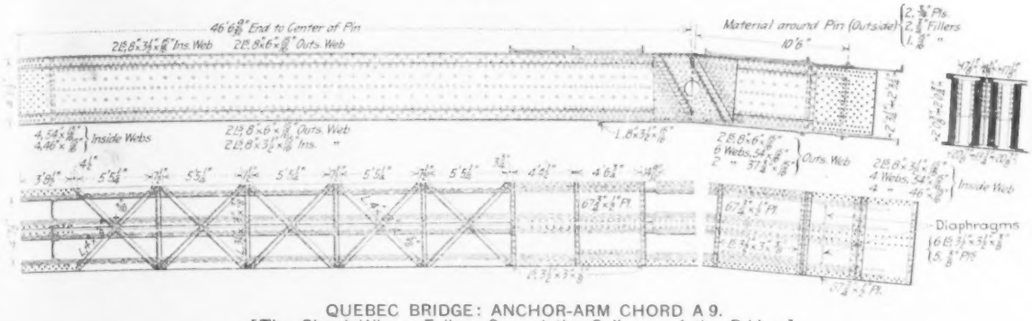
AN INTERNATIONAL EXPOSITION FOR ART and Building Industry will be held in St. Petersburg, Russia, from June to October, under the direction of the Society of Civil Engineers of Russia. The exhibition will have sections for the display of (1) building materials, (2) carpentry and metal work, (3) sanitary protection, (4) fireproofing materials, (5) electrical apparatus, (6) artistic furnishings, (7) literature and methods of technical instruction. The American agents of the exposition are Gerhard & Hey, Battery Park Bldg., New York City.



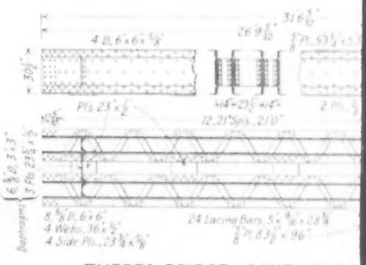
FORTH BRIDGE: TYPICAL CHORD.



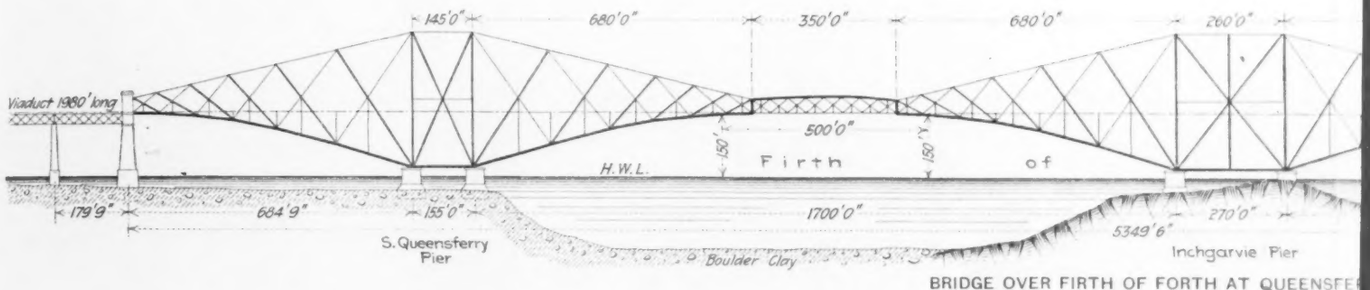
BLACKWELL'S ISLAND BRIDGE: ANCHOR



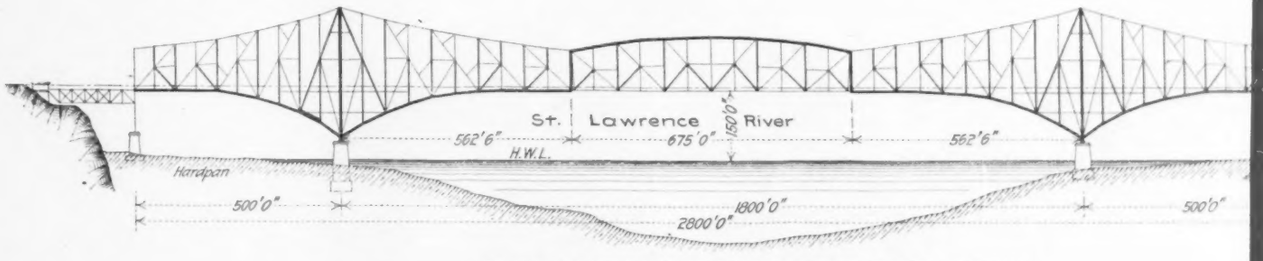
QUEBEC BRIDGE: ANCHOR-ARM CHORD A 9.
[The Chord Whose Failure Caused the Collapse of the Bridge.]



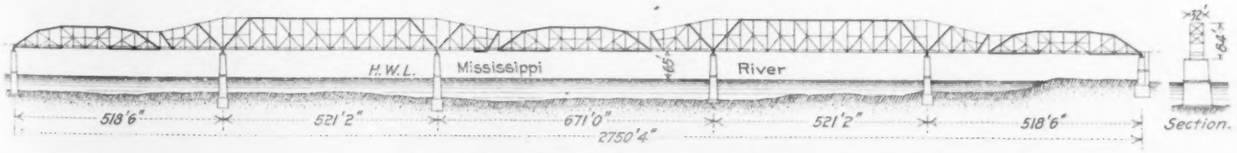
THEBES BRIDGE: CANTILEVER



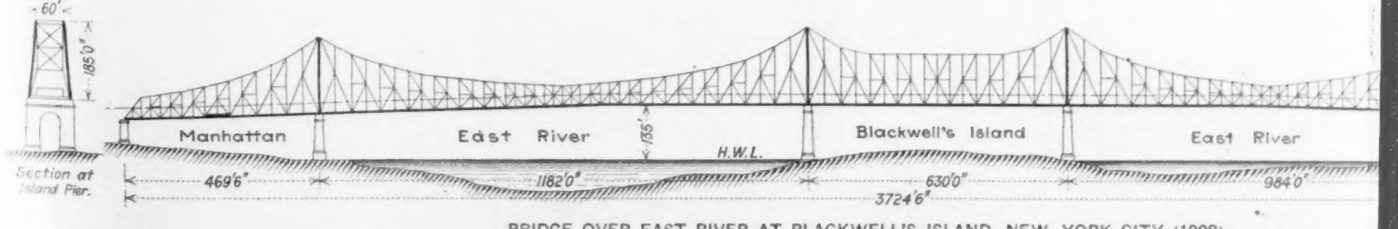
BRIDGE OVER FIRTH OF FORTH AT QUEENSFE



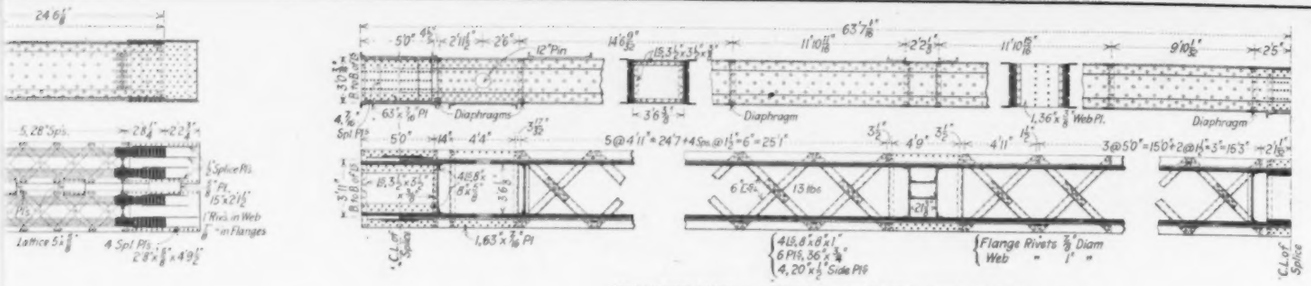
BRIDGE OVER ST. LAWRENCE RIVER NEAR QUEBEC, CANADA.



BRIDGE OVER MISSISSIPPI RIVER AT THEBES, ILL. (1903).

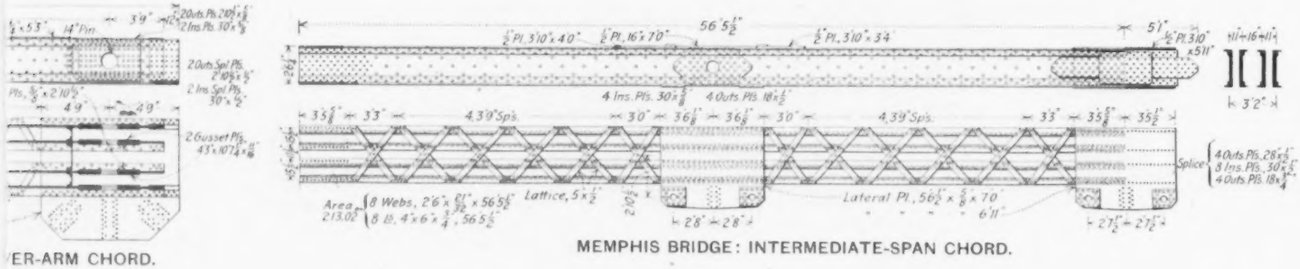


BRIDGE OVER EAST RIVER AT BLACKWELL'S ISLAND, NEW YORK CITY (1908).



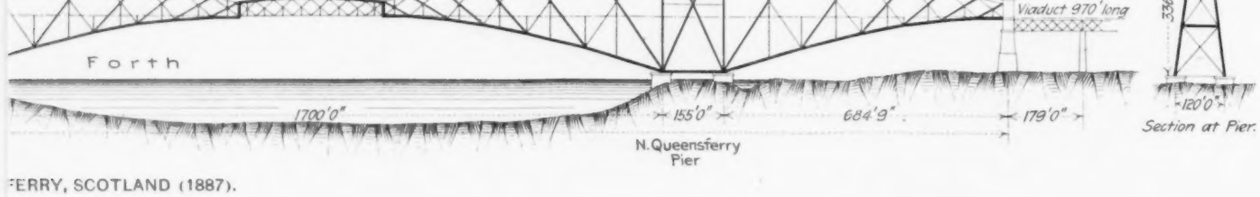
HOR-ARM CHORD.

MONONGAHELA BRIDGE: ANCHOR-ARM CHORD.

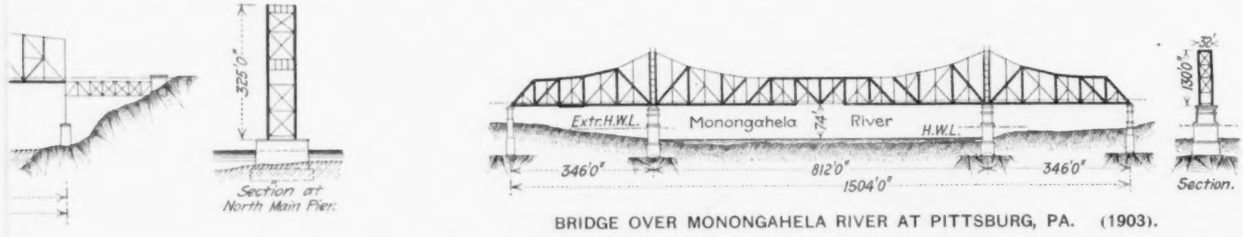


ER-ARM CHORD.

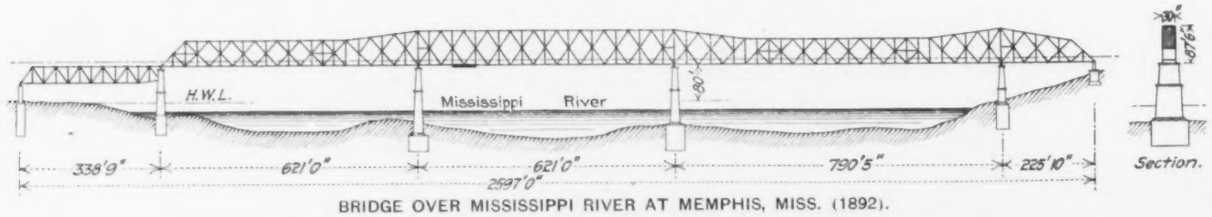
MEMPHIS BRIDGE: INTERMEDIATE-SPAN CHORD.



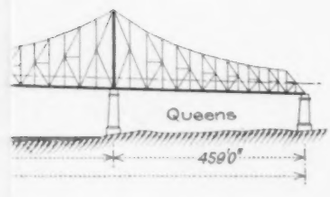
FERRY, SCOTLAND (1887).



BRIDGE OVER MONONGAHELA RIVER AT PITTSBURG, PA. (1903).



BRIDGE OVER MISSISSIPPI RIVER AT MEMPHIS, MISS. (1892).



THE SIX LARGEST CANTILEVER BRIDGES OF THE WORLD, AND TYPICAL DRAWINGS OF THEIR COMPRESSION CHORDS

[From the Report of the Quebec Bridge Commission of Inquiry]

NOTE: The outline elevations are drawn to a uniform scale: the detail drawings are similarly drawn to a uniform scale. Except in the case of the Forth Bridge, the chord shown in detail represents a particular member, located by a heavy line in the outline elevation.

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We complete with this issue our reprint of the Quebec Bridge Investigating Commission's Report. We have already commented on the excellent work done by this commission of engineers; but we particularly wish to emphasize at this time the important lessons which the engineering profession has learned from this great disaster. We need not take space to recount the many matters (in connection with the Quebec disaster) on which helpful light has been shed during the past eight months. They are fresh in the minds of all our readers who have followed the various articles in our pages.

But suppose there had been no investigation either by private enterprise or public authority—save the brief "coroner's inquest" farce which the law requires. Suppose that the Quebec bridge disaster had been treated as great disasters are usually treated in this country. Exploit the gruesome and sensational details for three days in the newspapers and then toss it one side and forget about it. Refuse all information to competent inquirers and if any public authority shows a disposition to set on foot an official investigation by experts, see to it that the proper wires are pulled which will prevent any such investigation being made. This is the common method of treating great disasters in the United States and it is conceivable that the calamity at Quebec might have been so treated. We can all see now how great the loss would have been to the engineering profession and to the public, had such a policy been followed.

The point we wish to emphasize, therefore, is that there should be more such official expert investigations of engineering disasters, and particularly where the disaster involves some unsettled point in engineering practice. Of course many accidents, even those involving heavy damage, happen from causes which are comparatively trivial and unimportant, and brief investigation would show that no important lessons were to be learned. On the other hand, an accident may result in little or no loss of life and yet investigation will reveal a hitherto unknown defect in design or material or administration, the knowledge of which will be of great importance to the profession.

And inasmuch as official investigations are as yet seldom held, the best that can be done is for engineers, through their professional journals to interchange the records of important engineering accidents. The work of Engineering News in this field has become so well known that engineers frequently forward to us voluntarily, accounts of the failure of engineering works in their vicinity. Good contributions of this sort to engineering literature are always appreciated, both by the editors and by the profession.

Do the engineers of the United States realize the value and importance to the profession of stream gagings extending over a term of years? If any engineer who reads this does not understand why long term stream gagings are needed, we suggest that he turn at once to the article by John C. Hoyt, Assoc. M. Am. Soc. C. E., on pp. 459, 460 of our last week's issue. Study carefully the tables of stream flow for a series of years given by Mr. Hoyt. Do engineers need accurate data as to the flow of streams? Of course they do. The question is of immense importance in almost every branch of the profession. Can an engineer find out for himself what the flow of a stream is, or is likely to be at the flood or at the low water period, and find it at the time when he wants to use the information? If any engineer thinks he can, we suggest further study of Mr. Hoyt's tables.

We do not say, of course, that an engineer will not desire to make some measurements of his own on any stream in which he is interested, as to its water power possibilities for example. What we do say, is that the information, given by gagings of typical streams all over the country, extending over a series of years, is of the greatest value to engineers and to the public.

In the second place, if you appreciate the usefulness of these gagings of river flow, lose no time in writing to the Representative of your Congressional district at Washington urging that the appropriation for the work of stream gaging be continued. Dispatches from Washington state that the Appropriations Committee have decided to cut off further money for stream gagings, and nothing but a general and emphatic protest from the engineering profession will turn the tide and enable the work to be continued.

We are aware that some engineers have criticized the work in stream gaging done by the Geological Survey. It has been said that the work ought to be done by some other body or done in some different way. Very likely there may be ground for criticism; in any large organization of which we have ever heard, some things are done poorly. But the point we want to bring to the attention of the engineering profession is the need for *team play*. All engineers agree that the work of stream gaging should be done, and practically all agree that the Federal Government is best fitted to undertake it. Surely then the thing to do is to get behind the organization that has already undertaken the work and made a fair amount of progress. It is easier to keep what one has gained than to make a new advance. If engineers fail to rally to the support of the stream gaging work, not only will the appropriation be cut off and the organization disbanded, but it will doubtless be a very long time before the work can be resumed.

A well-known hydraulic engineer in private practice has recently handed to us a paper prepared some months ago, before the question of abandoning the Government stream-gaging work came forward. A portion of it is so applicable to the present situation that we reprint it by his permission as follows:

Whenever the question of the sufficiency of a source of water for municipal and industrial purposes, for the development of power, for feeding a navigation canal or for irrigation arises, it becomes of paramount importance to know, not only the ordinary stream flow, but the range of flow, the minimum to be expected in a year, in a series of months or perhaps for days only, and the momentary maximum, which must be taken into account in designing structures.

Upon this information, the feasibility of projects for the utilization of the source of supply depends and upon it are based the types and dimensions of structures for the safe diversion and utilization of the required water.

Information of this class cannot be secured, in the field, by the civil engineer upon a moment's notice. He is not usually asked to make an investigation until the time has come when his conclusions are wanted. He will rarely, if ever, find the stream that he is to investigate at such stages that he can answer with confidence from his own observations all questions relating to low water, to high water, and to volume of flow; consequently he turns to the records, if there be any, of other examinations and measurements and uses these to amplify and verify his own observations. If there are no records to which he can turn for information concerning the past history of a stream, he may have to take recourse to the recollections of the oldest inhabitants, which unfortunately are far from reliable in such matters.

The importance of making a continuous record of this class of information has not been appreciated in our country until within recent years; and even at the present time those who are most insistent in their demand that it be collected and published, though it be often in advance of the actual exploitation of water resources, are generally supposed to have some ulterior motive behind their movement for the general good.

But no such motive is needed. The information wanted to-day should have been collected during the last 50 years and the information collected to-day will be welcomed 50 and 100 years from now by the engineers who are then entrusted with hydraulic problems. Moreover, the work of collecting information relating to the water resources of the country and to the water productiveness of certain regions that produce disastrous inundations is never complete, it must be continued indefinitely because the behavior of the stream and of the drainage basins which yield water are as varied as the seasons themselves. We may wait for centuries for the extremes of the climatic conditions on which the extremes of stream flow are dependent. These facts are known to the engineering profession, but the general public needs enlightenment.

The U. S. Geological Survey deserves commendation for the foresight which prompted the undertaking of such work by a scientific government bureau, and for the persistence with which the importance of this work has been urged year after year upon the Federal Government. The published results of stream gagings are of great value and every break therein is to be deeply regretted.

The funds placed at the disposal of the Survey have been inadequate and the records, acceptable as they are, do not therefore cover all parts of the country, neither could the work be done in all cases with the desired refinement. Moreover, work of this character is not appreciated by all members of Congress and the annual appropriations required for it are by no means assured.

It has been suggested, and it may be that the suggestion is a wise one, that the permanency of this work would be better assured if the hydrographic branch of the U. S. Geological Survey were elevated into an independent bureau. It should, in any event, be given authority (and the necessary funds) to make thorough investigations of water resources in all parts of the country, East, West, North and South. Full publication of this class of information will be of continuing benefit to the Nation. To this fact engineers must call attention as opportunity offers. Engineers are best qualified to testify to the loss that would be sustained by a neglect to collect and publish this information.

In these days when the public is more impressed with the novelty or picturesqueness of an off-hand opinion or impression, than with its relation to well-established principles, scientists with great names should be very careful what they say in public. This reflection is prompted by certain portions of Mr. Freeman's report on water-storage and water-power on the Genesee River, to which reference is made in two other places in this issue.

It seems that on the last visit of the late Lord Kelvin to this country, a year ago, he let fall some remarks at Rochester, N. Y., to the effect that the most feasible means of utilizing the water-power of the Genesee River drainage area and preventing floods on the lower portion of the river would be to construct numerous small dams and reservoirs on the head waters rather than one large one on the main river.

When Mr. Freeman took up his studies of the Portage water-storage and water-power project,

he found Lord Kelvin's remarks still so deeply impressed on the public mind, or some portion of it, that he felt it necessary to have studies made of a dozen relatively small storage possibilities, as well as of the single large scheme that had long been projected. The studies showed conclusively, Mr. Freeman states, the far greater economy of the single large reservoir.

This instance is not the only one of the sort associated with the memory of Lord Kelvin. At the opening of the Shoreditch combined refuse destructor and electric-light station a number of years ago, Lord Kelvin was one of the guests of honor. In his speech on that occasion, framed naturally in an optimistic vein, he expressed some sanguine opinions as to the possibility of lighting cities of the future with electric current generated from the heat of their burning refuse.

These remarks were quoted at the time in this country and have been repeated again and again, from that time to this, in discussions of the best means of getting rid of the refuse of American cities.

It is of interest to record that in the years that have elapsed, one of the most vaunted features of the Shoreditch installation, the thermal-storage adjunct, appears to have gone far towards dissolution, and certainly has not come into general favor. As to the real financial practicability of producing light from refuse burning at Shoreditch or elsewhere, complete and reliable figures of actual results are yet to be furnished.

Even if English experience were satisfactory in this field, it would still remain to be proved that the same thing could be done in this country where labor cost, fuel cost, and character of city refuse are all radically different from English conditions.

We may add further that a famous scientist may be a very unsafe guide in engineering matters. This is particularly the case if one takes statements made in some public speech where the obligation to say complimentary things may be uppermost, as was doubtless the case at the Shoreditch celebration, or things said when advancing years have dimmed the powers of the mind, as was undoubtedly the case with Lord Kelvin on his last visit to America.

The modified Passaic Valley trunk sewer plan, submitted to Paterson, Passaic, Montclair, Newark and other New Jersey communities last week, has led to renewed threats of litigation by the Merchants' Association of New York. Doubtless a considerable number of members of the association think they are rendering New York a great service in attempting to bar New Jersey communities from doing what New York City is itself doing—utilizing the vast tidal flow of New York Bay as a means of sewage disposal by dilution.

It is difficult to see, as long as New York and New Jersey front on opposite sides of the great tidal estuary, the lower Hudson, why the latter State has not as good a right as the former to discharge sewage into it.

As for any real damage to be done to the public health by such discharge as is proposed, the weight of the best engineering opinion is that there will be no pollution of the harbor waters at all comparable with what has been going on for many years in the discharge of sewage into the heads of the slips between piers. As a matter of fact, the Passaic Valley trunk sewer, with its proposed multiple-discharge outlet 40 ft. below the surface, is well-nigh the first attempt to use the harbor waters for sewage disposal in a carefully-planned, scientific manner, in place of the haphazard, easy-go-lucky methods of New York City.

Socially and commercially the New Jersey cities in the Passaic Valley are really an integral part of New York City. Any measure for their benefit really benefits the greater city. New York has sanitary work for its own needs before it, so important and so extensive that public spirited organizations like the Merchants' Association would be wise to interest themselves in that work rather than pursuing the will o' the wisp of excluding all sewage from tidal waters.

STATE WATER CONSERVATION AND UTILIZATION IN NEW YORK AND ELSEWHERE.

The detailed report on two possible large water-storage and water-power projects for execution by the State of New York, by Mr. John R. Freeman, M. Am. Soc. C. E., abstracted briefly elsewhere in this issue, makes pertinent some observations on the development of ideas and practice in the matter of State water conservation and utilization.

The first efforts at conservation of a natural resource which has seemed so abundant until recently as to require little care, were directed to the preservation of the purity of municipal water supplies. In the early days, these efforts were chiefly in the nature of legislation prohibiting pollution, with few or no measures for making the prohibition effective. Later on, as the pollution became greater and demands for larger volumes of water also increased, some of the most progressive of the thickly populated States supplemented their prohibitive legislation by investigations to determine the actual conditions of the waters being used for public supplies, with a view of ascertaining just what was necessary for their protection. Still later, the supervision of the discharge of municipal and other wastes into streams and lakes was undertaken by a few States, but recently the Ohio Legislature gave its State Board of Health control over the operation of both water and sewage-purification works, as described and discussed in our issue of April 16, 1908.

Naturally incidental to the conservation of the purity of streams has been the determination of the quantities of water available for public supply from a given drainage area, and the apportionment of the supply between different communities seeking to utilize it. Comprehensive investigations of available quantities have not yet been carried very far by any of the individual States, outside of New Jersey, but in a few instances something of the kind has been done, either independently or in conjunction with the U. S. Geological Survey. The investigations of the latter, however, are as yet, as they promise for some time to continue to be, the chief source of information as to the yield of surface supplies.

Massachusetts, through its legislative committee on water-supply and sewerage, has long retained a large measure of control over the appropriation of water for the supply of the municipalities within its borders. In exercising this control, the committee named relies largely upon the advice to cities and towns given by the State Board of Health. This advice is given in accordance with legislation of long standing, providing that plans for new water supplies must receive the approval of the Board before execution. It is practically useless for a municipality to go to the Legislature for authority to sell bonds and build works for additional supply unless it can show that its scheme has been approved by the Board.

In recent years, New York, New Jersey and Pennsylvania have each created State water-supply commissions, the primary purpose of which is to pass upon plans for new municipal water-supplies, with a view of protecting the interests of all the communities naturally dependent upon a given drainage area, and also with a view of protecting the rights of land-owners whose property may be required for water-supply purposes, and the further and less tangible rights of people whose occupation and earnings may be affected by the wiping out of industries and villages by the erection of large storage reservoirs like the Nashau, for the supply of Boston and vicinity, and the Ashokan, now under way in the Catskills for New York City.

In two States, New York and New Jersey, somewhat broad studies of water resources have been made in years past. The New York studies were first and naturally connected with the provision of water for the Erie and the Champlain canals, also taking into account for 40 years past the improvement of navigation in the Hudson River as well. During the past dozen years these New York studies have been broadened in scope and deepened in detail in order to determine water-storage possibilities for [illegible] purposes, flood-prevention and water-power.

The New Jersey studies, as carried [illegible] for a considerable number of years by the [illegible] Geological Survey, finally resulted in a valuable report on the water-supplies of the State, which, owing to its being the first report of its kind, and also owing to its breadth as regards consideration of rainfall, run-off and other factors influencing stream flow, has perhaps been of as great use outside as within the State. This report, it may be noted with emphasis, [illegible] into account the water-power possibilities of the State, possibilities far too generally and too long neglected by the commonwealths of the nation.

While considering New Jersey, the fact should not be overlooked that that State just missed anticipating by many years all the other States in conserving and developing the natural waters of the State in the interests of the people as a whole, instead of allowing them to be monopolized for private gain. A State Water Commission was created many years before the present State Water-Supply Commission was established, and submitted an important report showing how a joint water-supply for a large number of municipalities might be provided to the great advantage of each and all of them. The Commission urged State construction of storage and other necessary works, and contemplated making investigations for the benefit of other portions of the State. But all its work came to nothing, and the Commission itself finally passed out of existence through the failure to make appropriations for it and to appoint members to take the places of those who died or moved out of the State.

This unfortunate ending to plans that might have been of inestimable value to the people in New Jersey was due to the fact that private interests wished to establish and profit by a monopoly in the only water-supplies available for large and rapidly growing sections of the State, instead of having the monopoly vested in the people themselves. The opposition to State activity in water-supply matters went so far as actually to kill, year after year, all bills introduced in the Legislature that would have effectively provided for the prevention of stream pollution. When the water syndicate had closed contracts with the most important of the municipalities which it wished to supply, legislation against pollution began to be enacted and was subsequently made more and more effective. We hardly need say that this incident should be borne in mind by those who are now working for State water-supply measures designed in the interest of all the people, whether in New Jersey or elsewhere.

Going farther afield and taking a broader outlook from quite a different point of view, we see that some years ago a considerable number of the Western States, located in the region of deficient rainfall, made efforts to provide water for irrigation. The earliest of these efforts were directed to the formation of irrigation districts, which hardly come within the scope of our discussion, since they partook largely of the nature of municipal enterprises, and that under very limited State supervision. The demand for such districts was found in the failure of the irrigation companies to meet the needs of irrigators. This failure was due in part to overreaching efforts for the benefit of the companies, regardless of the interests of the irrigators, but very largely to lack of data as to water-supply possibilities and needs, and also to the fact that many of the water-storage works required for the development of potentially highly-fertile irrigable lands were beyond possibilities of private capital because of the many years that would be needed to put them on a money-paying basis.

After the irrigation districts proved inadequate to meet the needs of the Far West, State action was attempted under the terms of the so-called Carey Act. This act provided for the assignment by the United States of 1,000,000 acres each of land to such of the States as would agree to develop water-supply and irrigation works for the benefit of those areas. Although Idaho has carried out one or two notable developments

under the Carey Act, and, although something, we believe, has been done in one or two other States, this plan also failed to fully meet the needs of the arid region.

The next great step in making possible the utilization of the water resources of the Far West was the act of Congress under which the Reclamation Service is carrying out the many projects, some vast in extent, which have been noted from time to time in this journal (see Eng. News, April 23, 1908, for statement of progress on these various enterprises).

We have not space, and it is not our purpose at this time, to enlarge upon the work of the U. S. Reclamation Service. It is mentioned merely for the purpose of calling attention with emphasis to the fact that public funds, either National or State, must be utilized to make the most of the water resources of the country. In one portion of the country, water for irrigation may be of paramount importance; in another, water for municipal supply; in another, water for power; in another, water for navigation; while in some sections water for two or more of these purposes, which can be provided for the present only at public expense, may be imperative.

It was in recognition of the broad general facts just stated, and of the relation of the natural waters of the country to flood damages and their prevention, that President Roosevelt appointed the Inland Waterways Commission, which made its preliminary report not long ago. This Commission, obviously, has been devoting itself to the Interstate and National phases of the subject of water resources, which, as our readers understand, are many and complicated. In fact, so many and complicated are the problems which deserve attention at the hands of the permanent waterways commission, or better yet, the permanent commission on all the natural resources of the country which it is to be hoped will soon be provided for by Congress, that it must necessarily be some years before any definite plan of action can be formulated for execution by the general Government. Meanwhile, and in fact quite regardless of what such a commission or the general Government may or may not do, there is much that can and should be done by the several States. Eventually some of this work will doubtless be done in conjunction with the United States, and unquestionably some of it will be done in cooperation with municipalities.

The New York State water-storage investigations, outlined elsewhere in this issue, may well serve in many respects as a model for other State investigations, whether such investigations relate to water-power, municipal water-supplies or other undertakings. The point we particularly wish to bring out here is that instead of following the practice of most of the earlier New York investigations and attempting to spread a relatively small appropriation over the whole State, the New York State Water Supply Commission and Mr. Freeman applied the available money to two large and promising projects, which could thus be studied in detail. This was contemplated by the spirit of the act providing for water-power investigations, although not made possible by the appropriation. That is to say, the spirit of the act was to the effect that investigations should be similar in character to those that would be undertaken by an individual or private company who wished to determine the wisdom of putting private funds into the projects, but the appropriation was wholly inadequate to cover the State.

A somewhat similar detailed practical investigation was made in New Jersey by the Passaic Flood Commission since superseded in effect by the State Water-Supply Commission. The New Jersey studies, as has been stated recently in these columns, resulted in the recommendation for an immense storage reservoir on the Pompton River, one of the two tributaries of the Passaic. This reservoir, it is believed, would prevent floods in the Lower Passaic valley, while at the same time making available a large volume of water which will soon be greatly needed for municipal supply. It would also provide for a much-needed increase in the dry-weather flow of the Passaic below Paterson, thus improving sanitary conditions by affording greater dilution for the sewage discharged into the Lower Passaic, and, still further, it would add to the amount and greatly increase the reliability of the water-power at Little Falls, Paterson and Passaic.

This seems an appropriate place to note that there is scarcely one of the notable recent water-storage projects that would not make it possible to utilize water in more than one way. Thus, the great Sacandaga Reservoir would provide a large amount of water-power just below the dam forming it, increase by many thousands the horse-power available at 13 existing lower sites, and greatly improve navigation in the Hudson River. The proposed Genesee River Reservoir, also in New York State, would do the same general thing in the way of water-power provision, and would save a large amount of flood damage at and near Rochester. Each project would be of sanitary advantage through providing greater dilution for sewage during dry weather.

It should also be noted that State and National action and the expenditure of public funds will be necessary at many points sooner or later in order to insure the full utilization of water resources to the best public advantages, since the natural and unavoidable tendency of private capital, as suggested by the latest New York report, is to take the heart out of every proposition and leave the carcass to go to waste. In other words, the State or Nation can wait longer than can a private company for returns on its investments, whenever, as in the case of developing water resources, there are many public advantages incidental to the work. In contrast, private capital must have immediate returns and is anxious to secure large profits, and, therefore, acts chiefly for the present and considers the public good, if at all, almost solely as it affords a means of private gain.

Finally, one of the things brought out clearly by Mr. Freeman's report is the need for more data on all matters pertaining to plans for the full utilization of the water resources of the country. This need doubtless applies with even greater force to other projects and in other States, in view of the fact that for years past water-supply investigations have been carried on in the territory covered by Mr. Freeman's studies. Whatever the various States of the Union may consider feasible or otherwise in the way of the actual construction of water-storage, water-power and flood-prevention or other works, there would seem to be no doubt as to the wisdom of providing ample funds and employing the best men available to collect data showing their available water resources and all the benefits that would flow from their conservation. Each State should know these possibilities, if only to see that it wisely guards all the interests of all its citizens in granting the use of these resources to private individuals and corporations.

LETTERS TO THE EDITOR.

Rapid Work on the Raton Tunnel.

Sir: Under the heading, "Records in Rock Tunneling," you give in your issue of April 2 some data as to speed in which tunnel headings have been driven in this country, and also in Europe. In connection therewith, it may be of interest to your readers to have the record of the recently completed tunnel of the Atchison, Topeka & Santa Fe Ry. through the Raton Mountains, between Wootton, Colo., and Lynn, N. M.

March 25, 1907, Began excavating approaches.
May 9, 1907, South portal reached.
June 9, 1907, 412 *ft. heading driven.*
July 9, 1907, 788 feet heading driven, and connected with heading No. 5, from shaft No. 2.

Length of tunnel	2,790 ft.
Average distance per month driven in heading No. 6 for two months	394 ft.
Average progress of all headings, per month	697.5 ft.

Two shafts were sunk, and six headings were used, but owing to scarcity of men, only four were worked steadily, the other two being driven a day or two at a time, when help was plentiful. The heading contained about 150 sq. ft. The total excavation in heading was 14,332 cu. yds.

Bench was started at South end July 12, 1907, and completed December 23, 1907, that is in 5 months, 11 days. The greatest length of bench taken out in one month was 590 ft. Total yardage of bench, 43,080 cu. yds. A steam shovel was used in taking out the bench.

Heading No. 6 was in soft sandstone, the others in shale and coal. Drilling was done with pneumatic augers. The tunnel was lined with concrete, progress in lining being as follows:

Began putting in footings	Dec. 1, 1907
Began turning arch	Dec. 1, 1907
Lining completed	April 14, 1908
Total concrete	12,600 cu. yds.

The total excavation in approaches was 68,000 cu. yds. The tunnel was for second track and the portals were reached by cross cutting from the old line.

The section of the tunnel is 16 ft. wide inside of lining at top of rail, 17 ft. wide at a point 16 ft. above top of rail, and has a semicircular top; the clear height from top of rail to crown of arch is 24.5 ft.

The amount of timber used in temporary lining in the heading was 281 M. ft. B. M., and a total of 655 M. ft. B. M. was used for this purpose in the whole job. This does not include form lumber.

C. A. Morse,
Chief Engineer, A., T. & S. F. Ry. Co.

Topeka, Kan., April 20, 1908.

[So far as our information goes, the advance of 412 ft. in heading No. 6 in May, 1907, is exceeded among American records only by the Gunnison record of 449 ft. and the Sutro tunnel figure of 417 ft. It is specially noteworthy that this rapid advance was secured on so short a tunnel, as the high records of European work all were made on long tunnels, for which elaborate plant and well-organized working forces can be provided.—Ed.]

Answering Advertising: The Need for Standardized Log Rules.

Sir: May I have an opportunity to say a word which has struck me every time I have had occasion to advertise for men in your columns?

Recently I advertised for a draftsman, able to keep set double entry books, asking for references and *sample of work.* I had over 100 applications for the position and *not over ten per cent. sent samples* of work. If applicants for a position knew what an impression is made by a short, concise statement of qualifications and salary perhaps they would be more business-like.

May I also call your attention to the fact that two log rules have been devised which are accurate and which make all necessary allowances, which are scientifically elaborated and are practical: the International Log Rule, proposed by Prof. Judson F. Clark, Forester of the Department of Lands and Forests, Ontario, and the Universal Log Rule, of Profesor Daniels, Bulletin 102, Vermont Agricultural Experiment Station. The time is certainly ripe for someone to start a movement to do away with the old and inaccurate rules and to get the persons interested together to use one of the above.

Full information may be found in "Forest Mensuration," by H. S. Graves, published by Wiley & Sons, New York. I have used the International Rule in a report a short time ago.

Very respectfully yours,
Ellwood Wilson,
Forest Engineer, Laurentide Paper Co.

Grand'Mere, P. Q., April 17, 1908.

Sir: Anent Mr. Morse's article on "Log Scales" in your issue of April 9, several years ago at Minneapolis, Minn., some question came up on this matter. I wished to find out how they figured the feet board measure in a log. Right there in the center of the biggest milling industry in the world there was not a bookkeeper, a scaler or a proprietor that had been in the lumber industry all his life that seemed to have any idea as to how the amount was arrived at. They could only say they got it from the scale stick. Nor could I find any literature that helped me out. They all knew that they got in lumber about 30% over the log scale. As near as I could come at it while the methods of manufacture have all been changed the scale rule of 40 years ago still obtains. It would seem as though there ought to be a uniform rule applied.

E. T. Abbott.

Thrall, Cal., April 17, 1908.

Notes and Queries.

In our issue of March 26, 1908, p. 331, we said, in connection with the peculiarly constructed arch bridge across the Guindy in France that

the bridge consists of a three-hinged arch with the haunch hinges some distance up the ring, a common enough design in masonry, but unique, so far as we know, in steel arches.

We are since in receipt of a letter from M. Réné Bonnin, of Paris, calling our attention to the Austerlitz Bridge across the Seine for the Metropolitan Railway of Paris, which bridge was described by him in Engineering News, Dec. 7, 1905, p. 604, and which is a steel arch provided with hinges some distance up the arch ribs. The Austerlitz Bridge, however, has not the awkward bracket truss of the Guindy arch and it is built entirely of steel and not of a combination of steel and concrete as in the latter.

NEW YORK STATE WATER-STORAGE AND WATER-POWER INVESTIGATIONS.

Water-power as a possible valuable asset to the State of New York was recognized by the Legislature of 1907, in the passage of a bill (Chap. 569), directing

the State Water Supply Commission to devise plans for the progressive development of the water-powers of the State for the public use, under State ownership and control, and making an appropriation

of $35,000 for the purpose. In accordance with the terms of the act, the Commission engaged Mr. John R. Freeman, M. Am. Soc. C. E., of Providence, R. I., to take up the work. A progress report on the investigations to the close of the year 1907 was submitted to the Legislature in February, 1908, and has just been made public. This report consists of a brief statement by the Commission, followed by a detailed illustrated report of over 200 pages by Mr. Freeman.

The Commission states that the bill contemplated a final report giving "as complete and accurate information as would be required by a conservative business man who was looking only for profits before investing his fortune in a private enterprise." Obviously it was impossible, with the money available, to make studies of such a character of all the water-powers of the State. Accordingly it was decided to confine the investigations to the storage possibilities of the Sacandaga River at the junction of the Hudson near Conklingville, Saratoga County, N. Y., and to the Genesee River at or near Portage, N. Y., not far from Rochester.

The Commission calls attention to the fact that with proper storage of flood waters the rivers of the State, excluding Niagara and the St. Lawrence, might furnish at least 1,000,000 HP. agaginst 450,000 HP. now utilized. Estimating the advantage of water-power over steam-power at only $12 per HP. per year, this wasted energy is worth $6,000,000. To be added to this is $1,000,000 a year of direct damage caused by floods, besides indirect damage which no one can estimate.

The available water-powers of the State having been open to the public and easily acquired, all the most easily developed powers have already been utilized. Further development, for the most part, requires the storage of flood waters, and this, it is important to note, not only makes possible the development of power at the point of storage, but also increases the available power at all developments lower down the stream. The natural tendency of private capital is, of course, to "cut out of the work (of power development) everything that can be postponed or avoided, no matter how essential it may be to the future success of any well-considered plan." Co-operation of different interests is attended with difficulty, and there is always a lack of the power of condemning land for storage purposes.

The Commission believes that by building storage dams at State expense, damage by flood waters would be decreased, the larger resultant minimum stream flow would improve sanitary conditions, navigation in many instances would be improved, cheaper power for manufacturing purposes would be secured, various industries stimulated, employment of labor in existing manufacturing establishments made more continuous, and finally a "satisfactory annual income" be produced for the benefit of the whole State.

MR. FREEMAN'S STUDIES.

Mr. Freeman began his investigations on Aug. 6, 1907, and concluded them on Jan. 31, pressure of other work making his resignation necessary. Between the dates mentioned a number of parties were employed and in addition a number of engineers were engaged in special investigations.

THE SACANDAGA PROJECT.—It was found that there could be readily constructed on the Sacandaga a dam that would form a reservoir having a high-water area of about 40 sq. mi. or nearly the area of Lake George. The water thus stored would be so utilized as to increase the dry-weather flow at Mechanicville from 1,500 to 6,500 cu. ft. per sec. At the thirteen developed water-power sites from Corinth to Troy, inclusive, the water from this reservoir in the dryest month of a year of ordinary rainfall would add nearly 125,000 available horsepower on a 24-hr. 7-day basis; it would also make possible 20,000 HP. additional at undeveloped sites between Spier's Falls and Glens Falls. Or, if, instead of the dryest month, the six dryest months of the ordinary year be considered, 80,000 HP. on the same basis could be added. Mr. Freeman states that the new power thus provided at Hudson River sites "is far larger than the aggregate water-powers of Holyoke, Lowell, Lawrence and Manchester, N. H."

The estimates just cited do not take into account the power that could be developed at the reservoir itself. This development, it is suggested, might come at a later date, but should nevertheless be considered as the chief object of the dam and reservoir.

Estimates show that by building a power-house near Hadley, constructing a tunnel some two miles in length and deepening the last mile of the Sacandaga for a tail-race, a fall of nearly 200 ft. could be secured, thus providing from 25,000 to 30,000 HP. on the turbine shaft on a 24-hr. 7-day basis. If, instead, the power was more profitably used by concentrating it

into the ordinary working hours of the week, with a 40% load factor, loads aggregating upward of 60,000 HP., as measured at the consumer's end, allowing 85% efficiency for electrical conversion and transmission, could be supplied throughout a dry year, and transmitted in part for municipal purposes to Albany, Troy, Cohoes, Saratoga, or any of the communities within a radius of upward of 50 miles.

Mr. Freeman states that nowhere else within the State of New York, unless at Niagara, has he been able to find a site "which promises to be so well adapted to development under State ownership and control."

In the course of his investigations, Mr. Freeman seems to have well-nigh exhausted every possible source of information on the rainfall and run-off of the streams being studied. In addition to this, he had new rainfall-observation stations established and had special studies made of the old stations, in order to determine whether any conditions prevailed which would throw doubt on the accuracy of the rainfall record.

In writing of the evaporation from the large reservoir surface which it is proposed to create, Mr. Freeman stated that

the best data on evaporation yet available are those determined from the experiments of Mr. Desmond FitzGerald, made on the Boston water-works and published in 1889; but owing to the greater altitude and greater coolness of the Adirondack region, it is probable that the loss from the water surface of the proposed Sacandaga Reservoir

would be somewhat less than the 39 ins. annually determined by Mr. FitzGerald for the Chestnut Hill Reservoir near Boston. Results in Maine obtained by Mr. H. K. Barrows, Hydrographer, U. S. Geological Survey, during the past three years, show an evaporation of 29 ins. annually. Mr. Freeman accepts the average of the two figures thus given, or 34 ins., as the evaporation for the Sacandaga Reservoir, but from this he deducts 21 ins. as the evaporation from the present land surface, thus obtaining 13 ins. increased annual loss over the reservoir area caused by changing land surface into water surface. Assuming the average water surface as 37 sq. mi., the loss of 13 ins. would be equivalent to a constant daily flow of 36 cu. ft. per sec., which should be deducted from the mass curves used by Mr. Freeman in computing the yield of the Sacandaga drainage area. This is 2% of the estimated total yield (1,700 cu. ft. per sec.).

To Eastern engineers perhaps the most striking feature of Mr. Freeman's designs for the Sacandaga Reservoir is his proposal to build a hydraulic-fill dam founded on fine sand. Borings at the selected site indicated "that beneath the present river channel is a preglacial gorge perhaps 150 to 200 ft. deep, filled with fine sand." Mr. Freeman, therefore, contemplates building "a great earth dam of almost unprecedented thickness," which will be economically feasible by placing material by means of hydraulic fill. He states that, in deference to popular ideas, a concrete core-wall is added, and that, below this, triple sheet-piling would be sunk to a depth of perhaps 40 ft. He proposes to carry the dam 25 ft. above the flow line and to give it a thickness of 230 ft. at the flow line and of 1,000 ft. at the elevation of the present river bed, which is some 80 to 90 ft. below the proposed crest, besides placing a blanket of fine-grain material on the river bottom for some distance above the inner toe. In addition to all this, the plans provide for a spillway more than 800 ft. in length, with a straight, steep, smooth-course waste channel below.

Estimates for the cost of construction show an approximate cost of $5,000,000 for the dam and reservoir, and another $5,000,000 for the power plant, transmission lines, etc. The total fixed charges for the dam and reservoir, including interest at 4%, a sinking fund to redeem the original outlay in 50 years, an allowance for lost taxes to the towns affected by the flooding, and depreciation on the perishable portions of the work, would be $290,000. These charges distributed over the estimated 80,000 HP. available for six months in the year already mentioned, would amount to $3.75 per HP. per year. Not all the water-power owners would pay this amount at the outset, but some could pay very much more, so that Mr. Freeman concludes that a revenue could be derived which would give the State a fair profit on its outlay for the reservoir in addition to all the indirect gains that would result from its construction.

The fixed charges on the proposed power-house and transmission lines would amount to $128,000, or an average of $17 per HP. per year on a 24-hr. 7-day basis for 25,000 HP. On the more usual basis of the actual number of working hours per week day (say 11.2) and with a 40% load factor, the fixed charges would amount to $6.75 per HP., "which is only about one-third of the lowest cost of steam-power on a 10-hr. basis in large plants." Mr. Freeman suggests that cities within convenient reach might be supplied at wholesale at $10 to $15 per HP. per year, or .38- to .57-ct. per KW.-hr. on the 40% load factor basis, while other consumers might probably be secured at prices ranging from $15 to $25.

THE GENESEE RIVER PROJECT.—Inasmuch as the investigations made by Mr. Freeman were designed to show the possibilities, merely, of State water-power utilization, the somewhat detailed account of the Sacandaga project which we have given renders unnecessary going into like detail for the Genesee project. The Genesee scheme, however, has certain features peculiar to itself which are of no little interest. One of the primary objects of storage on the Genesee would be to prevent flood damages in and near Rochester. Besides this, the proposed site for the dam is in close proximity to Letchworth Park, a tract of land including a ravine and waterfall of great natural beauty, which has recently been given to the State of New York. The studies for the Genesee project contemplate an arched concrete masonry dam, which, aside from its arched form, is very similar in design to the Ashokan and Kensico dams for the new Catskill water supply for New York City. This dam would be some 137 ft. high above the present water level and 155 ft. above the approximate rock base. The whole design of the dam and reservoir is intended to add to rather than detract from the scenic features of the surroundings. It is proposed to maintain a flow over the existing falls which shall always be equal to the present dry-weather flow, at least during those hours when there would be visitors in the park.

To economize on the cost of sluiceway gates it is proposed to utilize the Venturi principle in the conduit leading to and from them, placing the gates at the throats. It is also proposed to operate the gates on the hydraulic principle, with water from the reservoir, the head of water on the gates and on the operating mechanism being thus always the same.

The project contemplates providing additional water-power at present developments, while at the same time utilizing sooner or later the head created by the dam.

A MUNICIPAL BREAD FACTORY is the subject of a call for bids issued by the city of Budapest, Austro-Hungary, closing May 2, 1908. A capacity of 55,000 lbs. of bread per day is desired, and the machinery is to be continuous. Machinery and ovens are to be furnished. Bids are in charge of Dr. Béla Molly, Budapest.

THE BREAK IN THE HAUSER LAKE DAM, MONTANA.

By F. L. SIZER.*

The Hauser Lake power plant is located on the Missouri River about 18 miles from the city of Helena, Mont. The dam is 70 ft. in height, 630 ft. in length and was built of steel bents supporting curved steel plates forming the water face of the dam. Both ends of the dam were founded on the solid bedrock, but about 400 ft. in the center section was on a gravel foundation. The dam was anchored at the up-stream toe to Friestedt steel sheet piling, driven in the hard river ground. There was also a rubble masonry fill at the up-stream toe located under the plates and down stream from the sheet piling.

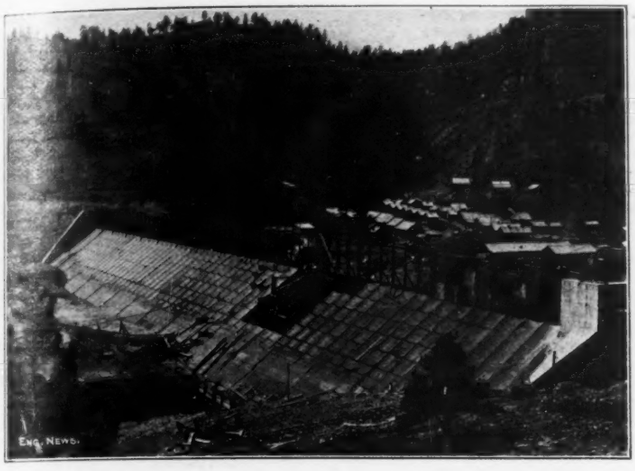


FIG. 1. THE HAUSER LAKE DAM NEARING COMPLETION.

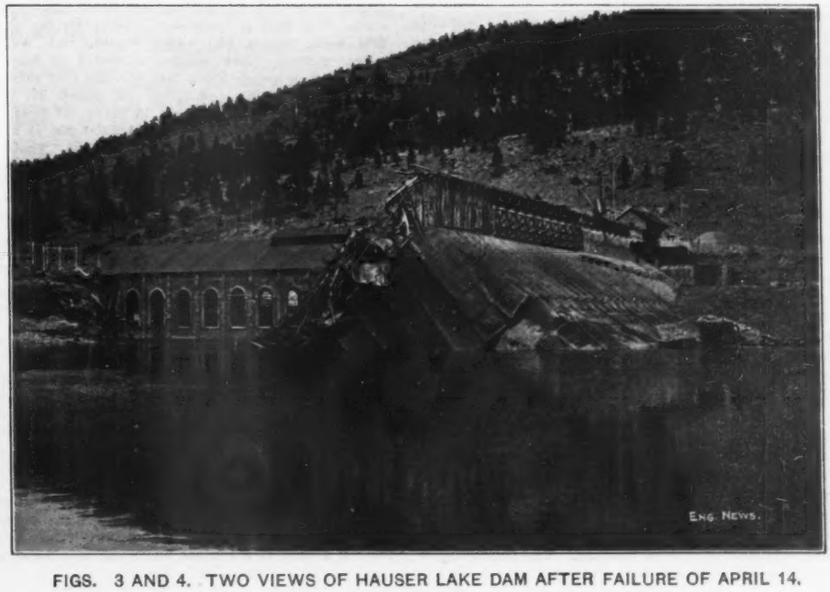
FIG. 2. THE HAUSER LAKE DAM BEFORE FAILURE, SHOWING LAKE COVERED WITH ICE.

The dam failed about April 14 at about 2.30 p. m. The initial break occurred at bent 30 about 400 ft. from the east or power-house end of the dam. The anchorage at this point apparently gave way, breaking the seal and allowing the water to pass under the rubble masonry fill. The water rapidly cut away the gravel, permitting settlement of this up-stream masonry and carrying down with it the lower end of the girder forming the upper member of the steel bent. The expansion joint in this girder and in the plates of the dam just above the top of the masonry, gave way, leaving the bents and plates unsupported in such a manner that the water pressure pushed over this section. About six minutes elapsed from the time the water first came through under the rubble masonry until the expansion joint failed and the first bent toppled over carrying out a section about 30 ft. in width. The tremendous rush of water rapidly widened this breach, the foundations on each side were undermined, and the posts and steel work buckled at right angles to the direction of the flow of the river. The bents continued to give way and fall until the breach widened to nearly 300 ft. The photographs show the break looking down stream after the water subsided. The break at the expansion joint referred to is plainly seen in the picture on the right hand side of the breach near the water level. The plates of the dam can be seen torn open along this line.

The power-house is of steel and masonry and contained five hydraulic and electrical units of 4,000 HP. each. The electrical power generated was stepped up to a pressure of 70,000 volts and was transmitted to Helena and Butte, the plant being operated in parallel with the plant of the Missouri River Power Co. located at Canyon Ferry, Mont., also on the Missouri River. The power-house and machinery sustained but very little damage. The electrical machinery is being dried out and it is estimated that the total loss to the power plant will not exceed $5,000.

Although much apprehension was felt, the flood did remarkably little damage, this being due largely to the fact that the river passes through canyons for 28 miles below the Hauser Lake plant, and the shores in this section of the river were uninhabited, the property belonging to one of the power companies. Below this point a few houses and fences were carried away and there was some damage to the track of the Great Northern Ry., but there was no loss of life. By the time the flood reached Great Falls, a distance of about 100 miles, it had been so distributed that the usual high water mark of the river was not reached and there was no damage whatsoever at that point. The total damage to property other than the dam, due to the flood, is estimated at about $30,000.

At the time of the failure the power was being used in the Butte mines and smelters, but the auxiliary steam plant located at Butte was immediately started up and this plant, together with the hydraulic station at Canyon Ferry, is



FIGS. 3 AND 4. TWO VIEWS OF HAUSER LAKE DAM AFTER FAILURE OF APRIL 14.

*Mining Engineer, Helena, Montana.

carrying practically all of the load, so there has been no serious interference with operations at Butte nor at Anaconda. Electric power for the city of Helena is supplied entirely from the Canyon Ferry plant, and for that reason there was no interruption of service in this city. The total damage has been variously estimated from $250,000 to $300,000, and the time to make the necessary repairs at about six months. The dam will be immediately repaired.

In addition to showing the condition of the wreck, one picture illustrates the dam during construction and another the dam after completion.

[A full description of the dam with working drawings furnished by the designer of the superstructure, Mr. J. H. Jackson, Assoc. M. Am. Soc. C. E., of the Wisconsin Bridge & Iron Co., was published in Engineering News of Nov. 14 last.

From the above description and from other information which has been furnished us, it appears clear that the original failure did not occur in the steel work of the dam but at the foundation. Water was seen flowing out under the apron and came running into the power-house from beneath the dam, heavily charged with silt, indicating the probability of a leak through or underneath the steel sheet piling. After five or six minutes of this heavy flow, the steel work apparently became undermined, and a section failed as stated above. It will be noted that the ends of the steel dam still standing which withstood the flood and the wrench from the falling portion are those which were founded on rock, only the section resting on a gravel foundation has gone out.

More or less difficulty was experienced in driving the steel sheet piling into the river gravel and the maximum depth reached was about 35 ft. It is also stated that the concrete around the heads of the sheet piling which supports the steel work at the base, was placed on some part of the dam in a depth of 10 ft. or more of water, so that its quality has been always a matter of uncertainty.

It may be added that this deep layer of gravel in the bottom of the Upper Missouri has given trouble before. The Canyon Ferry dam, referred to above, is founded on a similar gravel stratum and suffered partial failure some years ago.—Ed.]

THE RATING OF ELECTRIC FREIGHT LOCOMOTIVES.

The fact that the larger part of the art of electric traction has developed from trolley-car experiences and principles has led to the universal practice of rating electric railway motors in horsepower. This rating bears no relation to the maximum power which the motor can develop, but, according to a rule agreed upon a number of years ago, it represents that rate of working which, if continued for one hour, will heat the motor to the highest permissible temperature. As these motors are of the series type, they develop a higher power output if forced to run at lower speed, i. e., under greater load; and while they can carry the higher load for a short time, the permissible limit of heating is reached in much less than one hour.

In ordinary service, over an undulating profile or with frequent stops, the demands upon the motors average sufficiently to keep the heating below the one-hour limit in spite of temporary periods of much greater power output. The heating under the condition of maximum tractive effort is therefore rarely a limiting factor. In consequence the one-hour power rating is a satisfactory measure of service capabilities in ordinary work.

Mr. B. G. Lamme, of the Westinghouse Electric & Mfg. Co., recently called attention to a case where wholly different physical conditions prevailed, and continuous tractive capacity became the limiting factor. He suggests that it may be more useful for heavy hauling service such as was concerned in this case, to rate motors by the drawbar pull, which they can develop continuously without excessive heating. The following is an abstract of his remarks from the March Proceedings of the New York Railroad Club:

The Spokane & Inland Railway operates out of Spokane, Wash., a line of 116 miles in length. It is not one straightaway line of this length for there is one branch 40 miles long, the main line therefore being 76 miles long. The system is direct-current in Spokane but single-phase alternating current is used immediately outside the city, and right at the beginning of the alternating current part is an up-grade of 2% 8 miles long. In working out the original proposition all available data was obtained but as the line had not yet been laid out fully, complete information regarding the extensions could not be furnished. What information was given indicated that the extensions would, in general, be level or slightly rolling country. It turned out afterward that only about 10 miles of the entire road was level and about 40% of the total length represented grades of 1½% up to 2%. The general tendency is up-grade out of Spokane.

At first it was considered that the heavier service would be toward the city, the general trend of the grades thus being favorable. However, a rather heavy outgoing service has developed certain difficulties. The first electric equipment of this road covered 21 motor-equipped passenger cars, each with four 100-HP. motors per car, these cars to be operated either alone or with trailers, or in multiple unit. The order also covered six freight locomotives, each equipped with four 150-HP. motors, of the geared type. The motors had the usual one-hour ratings.

The locomotives were somewhat similar in appearance and construction to ordinary interurban cars, there being two swivel trucks, each carrying two geared motors. Each locomotive weighs 50 tons. With the gear-ratio used, each locomotive can develop about 15,000 lbs. tractive effort for one hour, at a speed of about 15 mi. per hr., and can develop continuously over 7,000 lbs. tractive effort, at about 21 mi. per hr. The 2% grade for 8 miles was considered to be the most severe condition, and as this required about 15,000 to 18,000 lbs. tractive effort for slightly over one-half hour the motor equipment, with its one-hour rating of 15,000 lbs. tractive effort, was considered ample for the service. However, on account of the numerous grades which were encountered it was found that the locomotives were worked too close to the maximum temperature. This was due principally to the fact that the method of ventilation first applied was unsuitable. The motors on these locomotives were artificially ventilated or cooled, as is now the regular practice of such equipments, but the air was supplied from blowers geared to the axles of the locomotives. This method of blowing is effective at high speeds but possesses the defect that the amount of air blown through the motors decreases very rapidly as the speed decreases with heavy load. In other words, the blowers deliver the least air at the time they should deliver the most. Also, no air is blown over the motors when the locomotive is at standstill. In order to overcome this difficulty separate blower outfits were supplied which operate at full speed continuously independently of the speed of the locomotive. This gave the required ventilation and the locomotives so equipped, are able to handle their service over the present line without overheating. However, on future extensions of this line more difficult conditions will probably be found in the way of very long continuous up-grades and it may be necessary for these locomotives to be operated with lighter loads on such service.

On a later order covering eight locomotives the question of more severe conditions was taken into account at start. The type of locomotive is very similar to those first built, having two swivel trucks, each with two geared motors. Each locomotive weighs 70 tons. The motors are so proportioned that with the gear ratio used, a continuous tractive effort of over 16,000 lbs. can be developed at a speed of about 15 miles per hour, while a one-hour tractive effort of over 25,000 lbs. at a speed of 11 to 12 miles per hour can be developed. These locomotives are *rated on tractive effort instead of horsepower*, and it is to be noted that the continuous tractive effort is slightly greater than the one-hour tractive effort of the old locomotives. From what I have said before regarding tractive effort required on the 8-mile 2% grade, it is evident that these new locomotives can do this service, at about the continuous tractive effort, and therefore these locomotives could handle this load on such a grade indefinitely. In other words, the normal service of the locomotives on the grades is made to conform with their continuous tractive effort and the one-hour tractive effort is simply a margin or emergency condition.

There is one important difference between a steam and an electric locomotive. In a steam locomotive there is a certain maximum tractive effort which it can develop, and this can be developed as long as desired without detriment. In an electric locomotive, however, the maximum tractive effort which can be delivered is usually far in excess of what it can safely develop without overheating the motors. Over quite a wide range, the losses in a motor and the tractive effort which can be developed, are almost in proportion to each other, and as there is a limit to the losses which can be allowed without overheating there is also a limit to the tractive effort which can be developed without overheating. As the heating is a function of the period during which the losses are developed it is evident that the motor can be rated at various tractive efforts, depending upon the duration of the run. It is evident that the motor can stand a greater loss and therefore develops a larger tractive effort for one hour than it could for 5 to 10 hours which would correspond to the continuous rating of the motors. Therefore if the hauls are short and the service intermittent the motors could be rated in a tractive effort corresponding to such service, while if the haul were of long duration the motors should be rated in tractive effort corresponding to such larger periods.

In freight service therefore the proper rating of the locomotive should be in terms of the continuous tractive effort which they can develop without undue heating, and any higher ratings for limited periods should simply be considered as emergency ratings. The horsepower rating of such locomotives should be incidental as the horsepower with a given tractive effort is a function of the speed and the speed in turn is a function of the voltage applied. Experience shows that with artificial cooling the continuous tractive effort rating can be increased from 50% to 80% in many cases, while the one-hour rating will be increased a very much less amount, and in motors where such a method of ventilation is applied in the best manner the continuous rating continues to approach fairly closely to the one-hour rating; that is, it may become 65 to 75% of the one-hour rating.

SECTION OVERBREAKAGE IN GUNNISON TUNNEL, COLORADO.*

A question of great engineering importance in connection with the construction of tunnels is the amount of overbreakage beyond neat-line sections necessarily, or otherwise, resulting in excavating. The determination of such overbreakage is of fundamental importance to engineers in estimating the cost of tunnel construction, as well as in the preparation of specifications covering the same. The cost of lining the tunnel and of handling the excavated material is greatly increased by large percentages of overbreakage. If the tunnel is to be lined with masonry, the quantity of this lining is greatly increased if the entire space between the finished tunnel bore and the excavated lines is required to be filled with masonry, thereby greatly increasing the expense. If, on the other hand, the lining is made of a definite thickness and the space behind it filled with tunnel debris, cord wood or other material, this backfilling likewise adds greatly to the expense of construction. In drawing up specifications covering tunnel work, it must always be recognized by the engineer that a certain amount of overbreakage will obtain, and the contractor should not be restricted to impossible limits in this respect. In specifying the thickness of lining, the manner of placing it and the method of backfilling around it, careful consideration should be given to the probable overbreakage.

Of no less importance is a knowledge of overbreakage to contractors. The contractor will always find it desirable to study specification requirements relating to lining, overbreakage and backfilling. He should also be informed on the question of overbreakage, so as to make a reasonable allowance therefor in his bid, both in connection with the transportation of the excavated materials and the quantity and placing of the tunnel lining as well as the nature of the backfilling and the cost thereof. An intimate knowledge of the laws of overbreakage, if there are any such laws, would have saved many contractors from financial ruin.

In the construction of the Gunnison Tunnel† considerable study has been given to the question of overbreakage, and some data have been collected, a portion of which is contained in this article. The method of determining the overbreakage on which these data are based was as follows: In all timbered sections the cross-section of the tunnel was determined by measuring

*Prepared for publication by F. W. Hanna, Engineer, U. S. Reclamation Service, Washington, D. C., from a report by E. L. Driggs, Junior Engineer, U. S. Reclamation Service, Montrose, Colo.

[†Uncompahgre Irrigation Project, U. S. Reclamation Service, Colorado. See Eng. News, Sept. 8, 1904, for illustrated description, and June 29, 1905, for illustrated account of a cave-in of the tunnel.—Ed.]

TABLE OF OVERBREAKAGE IN GUNNISON TUNNEL, COLORADO.

Material.	Class of timbering.	Number of sections averaged.	Overbreakage. Maximum. Area, sq. ft.	Maximum. Per cent.	Minimum. Area, sq. ft.	Minimum. Per cent.	Average. Area, sq. ft.	Average. Per cent.
Adobe and gravel	6	40	69	36	23	12	44.3	22.9
Adobe and gravel	5	20	71	40	26	15	50.3	28.6
Adobe and gravel	4	84	99	61	12	7	43.6	27.0
Shale	4 Mod.	11	123	77	53	33	82.5	51.2
Sandstone	4 Mod.	13	96	60	39	24	74.9	46.8
Conglomerate	None	56	32	35	1	1	15.5	16.0
Granite								

from the timbers to the line of breakage perpendicularly to the face of the timber in eleven or more different points. The timber section and the excavated section were then plotted on paper, and the areas determined by means of the planimeter. In rock sections the cross-sections were obtained by means of a Heller and Brightley Sunflower instrument, and were similarly plotted for the purpose of determining the areas. A number of such cross-sections were obtained for each character of material encountered in the tunnel and the average, minimum and maximum areas thereof determined. The overbreakage area was in all cases taken as the total area outside of the timber line in timbered sections and outside of the standard tunnel line in rock sections.

The materials encountered were shale, sandstone or conglomerate, shale overlaid with sand and gravel, wet adobe and gravel and granite. The character of the excavation and the method of timbering for different classes shown in the accompanying table are as follows: Class 4, shale, timbered and lagged; class 4 (modified), sandstone, timbered; class 5, shale overlaid with sand and gravel, timbered and lagged; class 6, wet adobe and gravel, timbered and lagged; drift, granite, part section, no timbering. The tunneling on which the data in the table are based was done mainly by the overhead driving method except that called drift in which the undercut drift method was used.

NOTES FROM ENGINEERING SCHOOLS.

COLORADO SCHOOL OF MINES.—The President of the Colorado School of Mines has announced that with the opening of the college year 1908-9 special courses will be given for graduates of technical schools, colleges and universities who desire to specialize in certain lines of applied science. These courses are not open to undergraduates. The list is as follows: Coal mining, coal testing, advanced general metallurgy, chemistry of rare metals, electrochemical analysis, radio-activity and spectroscopy, cements and clays, ceramic geology, metallography, microscopic petrography, practical astronomy, mine-plant testing, advanced mine and mill design, advanced hydraulic and gas-engine design, kinematical drawing of mining machinery, electricity applied to mining, precise surveying and geodesy.

All persons holding the degree of A.B., B.S., Ph.B., M.E., E.E., or an equivalent, will be admitted to a two-year course, leading to a degree of E.M. or E.Met., without preliminary examinations, provided their education and experience is such as to enable them to undertake the work.

UNIVERSITY OF WISCONSIN.—The Artisan's School will begin its eighth session on June 20, and, in connection with this, summer courses in allied engineering and professional branches have been instituted to meet the demand of engineers, contractors, advanced students, etc. Work is thus offered in chemical, electrical, hydraulic, steam and gas engineering, in machine design and drafting, in mechanics and testing materials and in shopwork. The work of the Artisan's School is in general along these subjects, but in much more elementary practice and adapted for persons having only common school training or practical experience.

A FLY-WHEEL ACCIDENT at Waukeegan, Ill., April 22, caused the death of two men, serious injury to two others, as well as a number of slighter personal mishaps, and an estimated loss to the plant of the North Shore Electric Co. of $135,000. Several other buildings were damaged. A piece of the wheel ripped a hole in the roof of the Hotel Edmund, a block distant, badly frightening about a hundred guests. The wheel was released by the burning of its connecting belt.

A COAL MINE EXPLOSION in Mine No. 1 of the Ellsworth Collieries at Ellsworth, Pa., resulted in the death of four men. Three others were seriously injured, and the escape of about one hundred others who were thrown to the ground by the concussion was accomplished with much difficulty. The accident presumably resulted from the ignition of an unsuspected accumulation of gas in a new entry.

A LANDSLIDE, burying half a hamlet, occurred April 26, near Buckingham, Ont.; 30 inhabitants of Notre Dame de Sallette were killed by the mountain-mass which, loosened by rain and melting snow, rushed down upon their homes.

THE MEXICAN RAILWAY main line, at Gargantau Siding, west of Maltrata, was the scene of a bad railway accident, April 24. In a collision between a freight and the second section of a special excursion train bearing pilgrims from the Shrine of Guadaloupe, 28 persons were killed and fifteen seriously injured.

TERRIFIC STORMS THROUGHOUT THE SOUTH and southwest of the United States on the night of April 23 and the morning of April 24 destroyed many towns in Texas, Iowa, Louisiana, Tennessee, Mississippi, Nebraska, Alabama, Georgia and Arkansas, and resulted in the death of many hundreds of people. The latest authentic list of casualties gives in Louisiana, Mississippi, Alabama and Georgia, 303 killed and 1,213 seriously injured, but the wide extent of the damage precludes any exact account in the remaining states. The devastated condition of many of the towns has occasioned the institution of martial law and organized relief work has been started under state supervision. The storms were preceded by long-continued heavy rains, and in many places the flooded rivers are adding considerably to the damage to property and discomfort to the residents.

A DISASTROUS COLLISION AT SEA occurred off the Isle of Wight April 25. The transatlantic liner "St. Paul" coming down the Solent in a dense snow storm struck the armored cruiser "Gladiator" of the British Navy amidships, cutting 20 ft. into her side, and compelling the foundering man-of-war to seek the shallows off Yarmouth, where she now lies a complete wreck. Damages to the "St. Paul's" bow were fortunately not severe below the water-line. After rendering all possible assistance to the disabled cruiser, she steamed back to port; none of her passengers were injured. The loss of life to the crew of the "Gladiator" was one lieutenant, five petty officers and 22 seamen; only the proximity of the shore, 500 yds. away, prevented a far more serious casualty.

THE SEPTIC TANK PATENT SUIT brought by the Cameron Septic Tank Co., of Chicago, against the Village of Saratoga Springs appears to have been brought to an end, so far as further trial is concerned, by the denial of the petition for a writ of certiorari by the U. S. Supreme Court. The next steps for settlement will be damage claims.

A REVISED PASSAIC TRUNK SEWER PLAN has been submitted to the several municipalities in the Passaic Sewerage District by the Passaic Valley Sewerage Commission. The general plan for the trunk sewer from Paterson to deep water in New York Bay is followed, but there are some modifications in detail. It is now proposed to use concrete throughout, unless it be for the multiple-disposal conduits at Robbins' Reef. The trunk sewer would start at Paterson and would range in size from 4 ft. at Paterson to 13½ ft. at the pumping station at Newark Bay. At a diameter of about 7 ft. the shape would change from circular to horseshoe as far as the pumping station at Newark Bay; but the tunnel, 12 ft. in diameter, from the pumping station beneath Newark Bay, the city of Bayonne, and New York Bay, to Robbins' Reef, would be circular. At the Newark Bay pumping station the sewer would be 26 ft. below low water. Sewage would here be raised some 40 ft. by means of five 60,000,000-gal. centrifugal pumps. The estimated cost of the project is $12,250,000. The estimated annual charges range from $170,000, when the flow is 120,000,000 gals. per day, to $283,000 for the full capacity of 360,000,000 gals. The cost would be apportioned, according to taxable values, among Paterson, Passaic, Newark, Montclair and other communities served. The several municipalities have 90 days to accept the plans and join the district, but if they do not join they must build independent sewage works. The engineering report of the Commission is signed and approved by the following engineers: Rudolph Hering and George W. Fuller, M.'s Am. Soc. C. E., Hydraulic and Sanitary Engineers, 170 Broadway, New York City; Allen Hazen, M. Am. Soc. C. E., of Hazen & Whipple, Consulting Engineers, 103 Park Ave., New York; William M. Brown, M. Am. Soc. C. E., Chief Engineer Metropolitan Sewage Works, 1 Ashburton Place, Boston, Mass.; and Edlow W. Harrison, M. Am. Soc. C. E., Civil and Consulting Engineer, 15 Exchange Place, Jersey City, N. J.

THE BALDWIN PRIZE for the best assay on municipal government, offered yearly for several years past by the National Municipal League in memory of the late Wm. H. Baldwin, has been awarded for 1908 to Mr. A. E. Pinanski, of Harvard College. The subject for competition this year was the relation of the municipality to the transportation service. Mr. Pinanski dealt with the Street Railway System of Metropolitan Boston. Honorable mention was given in three cases, as follows: Mr. Edmund W. Ogden, Harvard College, Class of 1910; Mr. C. A. Griffin, of the Junior Class of Missouri University, and Mr. George B. Rebmann, University of Pennsylvania, Class of 1911. The awards were made by the three following judges: Messrs. M. N. Baker, Editor of Engineering News; Robert Treat Paine, Jr., of Boston, and Henry Bruère, of the New York Bureau of Municipal Research.

THE PENETRATION OF THE RICKEN TUNNEL was accomplished on March 30, at 4.15 a. m., as noted in our issue of April 7, 1908. It appears that, as we then surmised, the progress made in the last month was all made in the south heading and exceeds any previous month's work in a single heading. It amounts to 531 ft. The work was done wholly by hand drilling, as throughout this tunnel. This is a new figure for hand-drilled tunnel progress, exceeding by 70 ft. the best previous month's advance of one heading in the Ricken tunnel. For machine-drilled rock tunneling, the Simplon tunnel record of 685.5 ft., the world's record, is far ahead. But no American rock tunnel has shown figures even as high as the Ricken tunnel record, the Gunnison tunnel record of January, 1908, 449 ft., with machine drills, being 82 ft. less than the new record of hand-drilled work in the Ricken tunnel. [See "Records in Rock Tunneling," Eng. News, April 2, 1908.]

The Ricken tunnel was begun over four years ago. Drilling was started in the north heading on Jan. 7, 1904, and in the south heading on Feb. 2, 1904. In the hard rock sections, ordinary hand hammer drills were used, but in the soft rock, which made up the larger part of the work, rotary drills similar to ordinary hand ratchets were employed. The tunnel is 28,221 ft. long between portals. The headings met 14,432 ft. from the south portal. As we noted at the time, fire damp was struck in 1907, which delayed the advance of the headings seven or eight months. A strike in 1904 also caused delay, so that the net time of working was about 3½ yrs.

THE ROCK CREEK VALLEY AT WASHINGTON, D. C., has been made the subject of a recent report by the Engineer Commissioner of the District of Columbia, under an appropriation of $4,000 made by Congress to be expended in an investigation of the proper treatment of the gorge. This valley runs between rather steep banks along the northwestern boundaries of the city of Washington, separating it at the upper part from a wooded suburban district and nearer the Potomac from the old town of Georgetown, now a part of the city corporation. Unfortunately the banks of the stream are for a greater part of the way under private control, which has had small consideration of the landscape possibilities of the topography so that long-continued dumping of refuse and ashes has made a most unsightly scrap heap of what might be made a remarkably artistic park. The recent investigations went into the subject of arching over and filling in the creek both in parts and as a whole, but recommended as the best solution an open-valley treatment in which the crossing streets are carried over the gorge on bridges and the entire slope down to the creek is developed as a city park with drives and roads and low level bridges. This procedure would involve extensive land condemnation and considerable grading, but would cost less than a closed through treatment, and would provide the city with a beautiful park running from the upper residential district down to the present riverside drive and speedway on the Potomac. The estimated cost of the recommended project is $4,750,000. The report will now be submitted to the federal Congress for further instructions.

PRACTICAL INSTRUCTION IN THE USE OF DIVING apparatus and appliances for artificial respiration at schools of civil and mining engineering was suggested by Dr. J. S. Haldane, F. R. S., during the reading on April 6 of a paper on diving before the North Staffordshire Institute of Mining and Mechanical Engineers. He instanced that the mining school of Bochum, Germany, where such a course is provided, was during the last year called upon 16 times to aid in repair or relief at subaqueous accidents.

PERSONALS.

Mr. Wm. Archie Weldin, C. E., Asst. to the Construction Engineer of the H. C. Frick Coke Co., has resigned.

Mr. Geo. H. Burgess, Assoc. M. Am. Soc. C. E., has been appointed Principal Assistant Engineer of the Erie Railroad Co., with office at 50 Church St., New York City.

Mr. Wallace Belcher, Jun. Am. Soc. C. E., with the firm of J. G. White & Co. of New York, has been sent to Colorado Springs as structural engineer for the Central Colorado Power Co.

Mr. Henry B. Drowne, Assoc. M. Am. Soc. C. E., at present an Assistant Engineer with the State Board of Public Roads of Rhode Island, has been appointed Instructor in Civil Engineering at Brown University.

Mr. B. B. Gordon, Chief Engineer of the Colorado Southern, New Orleans & Pacific and the Beaumont, Sour Lake & Western Rys., has resigned. Mr. C. W. Fisk, Engineer Maintenance of Way, with headquarters at Beaumont, Texas, will assume the duties formerly discharged by Mr. Gordon.

Mr. H. A. Foster, M. Am. Inst. E. E., Resident Engineer in Baltimore for L. B. Stillwell, has been transferred to the New York office. He retains supervision of the electrical and steam work at Baltimore, which duties will be shared respectively by Mr. H. S. Putnam, M. Am. Inst. E. E., and Mr. Van Vleck.

Mr. H. R. Burroughs, Designing Engineer for the Southern Pacific Ry. Co., has been appointed Engineer of Buildings for the Pacific Fruit Express Co., at San Francisco, Cal. Mr. Burroughs will make temporary headquarters at Las Vegas, Nevada, where he is engaged on the construction of a reinforced-concrete ice-plant building.

A bronze statue of the late John A. Roebling, executed by William Couper, of New York, will be unveiled next June at Trenton, N. J. The sum of $14,000 was subscribed by the people of Trenton for the erection of the statue, not so much in honor of Roebling's work as an engineer of great suspension bridges as for his activity in Trenton's industrial development, the John A. Roebling's Sons Co. having been founded by him.

Col. Washington A. Roebling, Pres., John A. Roebling's Sons Co., of Trenton, N. J., and Mrs. Cornelia W. Farrow, of Charleston, S. C., were married at Dalton, Mass., April 21. The ceremony took place in the home of the bride's school friend, Mrs. Frederick G. Crane. Col. Roebling, who is 70 years old, has for five years been a widower. Among the wedding guests were Mr. and Mrs. John A. Roebling, the former the son of the bridegroom, and Carl and Siegfried Roebling, of Trenton. Col. Roebling was early associated with his father, the late John A. Roebling, in his bridge construction work; on the demise of the latter his plans for the Brooklyn Bridge were brought to completion by the son.

Prof. Frank Soulé, M. Am. Soc. C. E., former Dean of the College of Civil Engineering, University of California, Berkeley, Cal., has by his own request been retired as the active head of the department and has been honored with the rank of Emeritus Professor of Civil Engineering. Prof. Soulé has been connected with the University for 39 years. He will be succeeded as head of the department by Prof. Charles Derleth, Jr., Assoc. Am. Soc. C. E., formerly Associate Professor, Structural Engineering, University of California. Mr. Derleth's contributions to Engineering News on the San Francisco disaster of April, 1906, will be remembered by many of our readers. He also served as a member of the Committee on Fire and Earthquake Damage to Buildings, of the State Earthquake Investigation Commission.

Obituary.

Herman Esser, retired Vice-President of the Keuffel & Esser Co., died April 16, at Godesburg, Germany, of pneumonia. He founded the business partnership with Mr. W. Keuffel in 1867, and for 35 years actively promoted the interests of the firm until, in 1902, he retired from business and took up his residence in Germany.

Chas. D. Pierce, M. Am. Soc. M. E., Consul General for the Orange Free State, committed suicide with illuminating gas in New York City, April 24, at the age of 60 years. He was President of the Pierce Well Engineering Co., of New York and Jersey City, and is said to have drilled artesian wells in nearly every country in South America.

John B. Harper, M. Am. Soc. C. E., Chief Engineer in charge of Indian irrigation works in New Mexico and Arizona, died March 24, at Los Angeles, Cal., at the age of 52 years, after a successful career as civil and mining engineer. The last four years of his life were given to the construction of the great Zuni dam on the Zuni Indian Reservation in New Mexico. He had been ill for a number of months, following an attack of pneumonia, and should have ceased work at a much earlier date, but was determined to complete the structure.

ENGINEERING SOCIETIES.

COMING MEETINGS.

AMERICAN ELECTROCHEMICAL SOCIETY.
April 30-May 2. Annual meeting at Albany, N. Y. Secy., Joseph W. Richards, Lehigh University, South Bethlehem, Pa.

AMERICAN WATER-WORKS ASSOCIATION.
May 11-16. Annual meeting at Washington, D. C. Secy., John M. Diven, 14 George St., Charleston, S. C.

NATIONAL ELECTRIC LIGHT ASSOCIATION.
May 19-22. Annual convention at Chicago. Secy., W. W. Freeman, 29 West 39th St., New York City.

NATIONAL FIRE PROTECTION ASSOCIATION.
May 26-28. Annual meeting at Chicago. Secy., W. H. Merrill, Jr., 382 Ohio St., Chicago.

AMERICAN FOUNDRYMEN'S ASSOCIATION.
June 9-11. Annual convention at Toronto, Ont. Secy., Richard Moldenke, Watchung, N. J.

MASTER CAR BUILDERS' ASSOCIATION.
June 17-19. Annual convention at Atlantic City, N. J. Secy., Jos. W. Taylor, 390 Old Colony Bldg., Chicago.

AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 22-24. Annual convention at Atlantic City, N. J. Secy., Jos. W. Taylor, 390 Old Colony Bldg., Chicago.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
June 23-26. Annual convention at Denver, Colo. Secy., C. W. Hunt, 220 West 57th St., New York City.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
June 23-26. Annual convention at Detroit, Mich. Secy., C. W. Rice, 29 West 39th St., New York City.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION.
June 23-26. Annual meeting at Detroit, Mich. (In connection with convention of American Society of Mechanical Engineers.) Secy., A. L. Williston, Pratt Institute, Brooklyn, N. Y.

AMERICAN SOCIETY FOR TESTING MATERIALS.
June 23-27. Annual meeting at Atlantic City, N. J. Secy., Edgar Marburg, University of Pennsylvania, Philadelphia.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
June 29-July 2. Annual convention at Atlantic City, N. J. Secy., Ralph W. Pope, 33 West 39th St., N. Y.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.—The next monthly meeting of the society will be held Tuesday evening, May 12, in the Engineering Societies' Building, New York City. A paper on "Clutches," with special reference to the types used on automobiles, will be read by Mr. Henry Souther, M. Am. Soc. M. E., President and Treasurer of the Henry Souther Engineering Corporation of Hartford, Conn.

THE CANADIAN CEMENT AND CONCRETE ASSOCIATION was organized in Toronto, Canada, April 20. It is modelled after the National Association of Cement Users of the United States and embraces the entire Dominion. The following officers have been elected: Pres., Peter Gillespie, Lecturer on Theory of Construction, University of Toronto; Vice-Pres., C. F. Puifer, Ideal Concrete Machinery Co., London, Ont.; Sec.-Treas., A. E. Uren, Toronto.

AMERICAN RAILWAY ASSOCIATION.—At the meeting of this association in New York City, April 22, announcement was made that its membership now consists of 338 roads and its associate membership of 44 roads, having a total of 242,400 miles in operation. Reports were submitted by committees on Safety Appliances and Standard Rail and Wheel Sections; the committee on Standard Location for Third-Rail Working Conductors has sent out a pamphlet giving the results of its work. The following officers were elected: F. A. Delano, President; W. A. Gardner, Second Vice-President; H. U. Mudge and J. Kruttschnitt, members of the Executive Committee.

IRON & STEEL INSTITUTE.—The annual meeting of the Institute will be held at the Institution of Civil Engineers, Westminster, May 14 and 15. The following papers will be submitted: "Cast Iron in the Construction of Chemical Plant," Mr. F. J. R. Carulla, Derby; "An Experimental Electric Furnace for the Smelting of Iron," Prof. B. Igewski, Kiev; "The Pyrometric Installation of the Ordnance Factories, Woolwich," Mr. J. Wesley Lambert, Woolwich; "Improvements in Plate Rolling Mills," Mr. A. Lamberton, Member of Council, Sunnyside Engine Works, Coathbridge, N. B.; "The Application of Color Photography to Metallography," Mr. E. F. Law, Assoc. R. S. M., London; "The Department of Metallurgical Chemistry in the National Physical Laboratory," Mr. W. Rosenhein, B. A., Teddington; "The Utilization of Blast Furnace Slag for Portland Cement," C. von Schwarz, Liege; "A New Fatigue Test for Iron and Steel," Mr. T. E. Stanton, D. Sc., Teddington; "The Physical Qualities of Steel in Relation to Its Mechanical Treatment," Mr. James E. York, New York. The institute has recently secured more commodious quarters in the same building heretofore occupied, No. 28 Victoria St.

MINING AND METALLURGICAL SOCIETY OF AMERICA.—This society, composed at present of about 120 members, was organized at the Engineers' Club, New York, April 20. It is to comprise mining and metallurgical engineers and geologists of North America, and is expected to occupy a field in professional activities entirely distinct from that of the American Institute of Mining Engineers. Membership in the society postulates eight years' practical or professional experience in the mining field, of which at least five years have been spent in responsible charge of work. Graduation from a reputable engineering school will be considered equivalent to two years' active practice. Annual dues are $10; an initiation fee of $25 will be required after the membership roll has attained the number of 250. Fifteen districts, each of contiguous territory and equal numerical representation as far as this is practicable, will geographically subdivide the continent, sending a total of 15 councilmen to the central governing body. This elects from its own number a president, two or more vice-presidents, a secretary and a treasurer.

Strong organization of the local sections for the purpose of social and friendly intercourse, as well as for the discussion of technical subjects, will be a prominent line of activity. A high ethical standard among members and among the profession generally is to be rigorously maintained. The members of the council are: Prof. H. S. Munroe, M. Am. Inst. M. E., Prof. of Mining, Columbia University; W. R. Ingalls, M. Am. Inst. M. E., New York; W. A. Lathrop, M. Am. Inst. M. E., Pres., Pennsylvania Coal & Coke Co., and F. Lynwood Garrison, M. Am. Inst. M. E., of Philadelphia; Robert H. Richards, M. Am. Inst. M. E., Prof. Min. Engr. and Metallurgy, Massachusetts Institute of Technology, Boston; Waldemar Lindgren, M. Am. Inst. M. E., Geologist, U. S. Geological Survey, Washington, D. C.; G. S. Rice, M. Am. Inst. M. E., Cons. Min. Engr., Whitebreast Fuel Co., of Ill., and Cardiff Coal Co., Chicago; L. S. Austin, M. Am. Inst. M. E., Prof. of Metallurgy, Michigan College of Mines, Houghton, Mich.; Dr. C. B. Dudley, M. Am. Inst. M. E., Chem., Pennsylvania R. R. Co., Altoona, Pa.; J. R. Finlay, M. Am. Inst. M. E., Colorado Springs, Colo.; J. H. Pratt, M. Am. Inst. M. E., Chapel Hill, N. C.; H. B. Patton, M. Am. Inst. M. E., Golden, Colo.; S. B. Christy, Prof. of Min. and Met., University of California, Berkeley, Cal.; T. A. Rickard, M. Am. Inst. M. E., Ed. "Mining and Scientific Press," San Francisco, Cal.; C. R. Claghorn, M. Am. Inst. M. E., Northwestern Improvement Co., Tacoma, Wash.

NATIONAL ELECTRIC LIGHT ASSOCIATION.—The thirty-first convention will open at the Auditorium Hotel, Chicago, May 19. Special sleeping cars will be attached to an express for Chicago leaving South Station, Boston, at 10.30 a. m., Sunday, May 17. A special train will leave New York City Sunday forenoon. Mr. G. F. Porter, 120 Liberty St., New York City, has charge of arrangements for this train and Mr. C. H. Hodskinson, 70 State St., Boston, Mass., will arrange for the Boston accommodations. The following program has been arranged:

May 18, evening. Reception in Banquet Hall.
Opening of Exhibit.

May 19, 10 a. m. President's Address.
Report of Committee on Progress.
Reading of Technical Papers.
Report of Committee on Grounding Secondaries.
2.30 p. m. Technical Papers.
Report of Committee on Uniform Accounting.

May 20, 10 a. m. Report of Committee on Meters.
Report of Committee on Gas Engines.
Technical Papers.
8 p. m. Executive Session.
Reports of Secretary, Treasurer and Executive Committee.
Report of Insurance Expert.
Report of Committee on Public Policy.
Report of Committee on Organization.
Report of Committee on Rates and Costs.
Election of Nominating Committee.

May 21 (Commercial Day), 10 a. m. Morning Session.
2.30 p. m. Report of Committee on Solicitors Handbook Prize Award.
Report of Committee on Cooperative Electrical Development.

May 22, 10 a. m. Technical Papers.
Report of Committee on Protection from Lightning.
Executive Session.

The following papers have been promised: "Distribution in Suburban Districts," G. H. Lukes, North Shore Electric Co., Evanston, Ill.; "Tape," Paul F. Lüpke, South Jersey Gas, Electric & Traction Co., Trenton, N. J.; "Series Incandescent Lighting with Tungsten Lamps," P. D. Wagoner, General Electric Co., Schenectady, N. Y.; "Power-Load Development for Central Stations of Moderate Size—Some Unappreciated Possibilities," J. R. Bibbins, Westinghouse Machine Co., East Pittsburg, Pa., and Chas. Robbins, Westinghouse Electric & Manufacturing Co., Pittsburg, Pa.; "Low Pressure Steam Turbines," J. W. Kirkland; "The Small Station and Its Economical Operation," J. T. Whittlesey, Public Service Corporation of N. J., Montclair, N. J., and Paul Spencer, United Gas Improvement Co., Philadelphia, Pa.; "Receiving Stations Operated from High-Tension Lines," S. Q. Hayes, Westinghouse Electric & Manufacturing Co., Pittsburg, Pa.; "Observations on the Precision of Different Types of Photometer," A. E. Kenelly and S. E. Whiting, Harvard University; "The Status and Commercial Possibilities of High Efficiency Lamps," W. W. Freeman, Edison Electric Illuminating Co., Brooklyn, N. Y.; "Illuminating Engineering," W. D'A. Ryan, General Electric Co., Schenectady, N. Y.; "The Value of Care and Maintenance of Meters," H. D. King, Public Service Corporation of N. J., Hoboken, N. J.; "Some Experiments in Combustion," S. J. Lenher; "Specifications for Construction of Joint Poles," Paul Spencer.

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